A PRACTICAL TREATISE

ON

Engineering and Building Foundations

INCLUDING

SUB-AQUEOUS FOUNDATIONS

ORDINARY FOUNDATIONS

THE COFFER-DAM PROCESS FOR PIERS,

HARBORS AND HARBOR WORKS

AND

DREDGES AND DREDGING

VOLUME I

ORDINARY FOUNDATIONS

[*Frontispiece*]

THE PONT DU GARD, NIMES, FRANCE.

A PRACTICAL TREATISE ON

ENGINEERING AND BUILDING FOUNDATIONS

INCLUDING

SUB-AQUEOUS FOUNDATIONS

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VOLUME I

ORDINARY FOUNDATIONS

FOURTH EDITION, REVISED AND ENLARGED

NEW YORK
JOHN WILEY & SONS, Inc.
LONDON: CHAPMAN & HALL, LIMITED
1920

PRESS OF
BRAUNWORTH & CO.
BOOK MANUFACTURERS
BROOKLYN, N. Y.

THIS WORK IS DEDICATED TO

Professor George C. Comstock

Director of Washburn Observatory

AND TO THE MEMORY OF

Professor C. N. Brown

Late Professor of Civil Engineering, Ohio State University

BOTH OF WHOM INCULCATED AN ABIDING DESIRE FOR THE APPLICATION OF MATHEMATICS AND THE SCIENCES TO THE PRACTICAL SIDE OF ENGINEERING

PREFACE TO FOURTH EDITION

THE preparation of a Fourth Edition of the writer's treatise on Sub-Aqueous Foundations has made it possible to expand the work into three volumes; and as it seemed desirable to add a large amount of data not only on the masonry of retaining walls, abutments and bridge piers, but on other classes of engineering foundations, including extensive data on similar construction for buildings, the title has been changed to the more comprehensive one of "Engineering and Building Foundations."

The first volume covers all of the original matter on piles, pile driving, and ordinary foundations, with the addition of much information on piles and pile driving in Appendix XI, that will be found useful in designing foundations on piling. The retention of considerable data on types of foundations that are seldom used in the more advanced sections of the world has been due to the fact that in newer sections of the United States and in new countries such methods continue to be used in the construction of foundations of a somewhat temporary character.

The very considerable information on the masonry of retaining walls, bridge abutments, and bridge piers, that has of necessity been given in various chapters, and the calculation of bridge piers and retaining walls as given in Chapter XII, has been extended in Chapter XVI on the design of retaining walls and culverts, in Chapter XVII on the design of bridge abutments, and in Chapter XVIII on the design of bridge piers. This makes the first volume a quite comprehensive treatise on ordinary foundations.

The use of reinforced concrete has somewhat revolutionized the character of engineering masonry, but when the cost of steel is high, and sand, gravel, cement and stone are low in cost, then solid masonry of concrete or stone may be found the cheapest to use in a given location. Certainly in many cases a heavy type of construction will often be found advisable from the standpoint of permanence alone.

The designer of engineering masonry should always consider most carefully the additions of small cost that can usually be made to

enhance the appearance of a retaining wall, abutment or pier, such as corbel courses, either plain or moulded, belt courses, and moulded base courses. Masonry should also be designed to harmonize with the other portions of a structure, not alone as to its details, but as to size and general proportions. This feature is most often overlooked in designing the skewbacks of large steel arches, and while all thrusts and stresses are provided for, the masonry is made too slender for proper appearance. Attention need only be called to the excellent solution found for Hell Gate Arch, to realize that appearance, harmony, and economy can all be provided for in the most simple and reasonable manner.

The design of metal and concrete caissons, open dredged caissons, pneumatic caissons, ballmatic caissons, and deep foundations generally has been covered in a comprehensive manner in the second volume. The methods used for building foundations, the loads on them and their proper distribution has also been covered in this volume, together with methods of underpinning, mill building and machinery foundations.

The foundations for dams, sea walls, dry docks, and locks, have been incorporated in the same volume, together with methods of concreting, forms for concrete work, and methods of estimating the cost of foundation work, and their immediate superstructures.

The matter pertaining to piers and wharves, clam-shell dredges, dipper and ladder dredges, suction and hopper dredges, tug boats and scows, divers and diving, rock fill foundations and quarries, has been incorporated in the third volume, which has been expanded to cover the design and construction of harbors, of which a large part of the construction is directly related to foundations. The new matter in this volume covers large clam-shell, dipper, and suction dredges; dredging pump design; output of dredges; harbor tunnels; sea-going tugs; large scows; and concrete scows.

The immense amount of work involved in the preparation of the new chapters will, it is hoped, receive repayment in the service such data will be to the engineering profession.

CHARLES E. FOWLER.

NEW YORK CITY,
January, 1920.

INTRODUCTION

The greater part of foundation work is of an ordinary character. And while difficult foundations have been quite fully treated by engineering writers, ordinary ones have too often been passed over with mere mention, or treated in such a general way that the information proves of little value in actual practice.

Many valuable examples of work of this character have been described in current engineering literature, and it is hoped that by bringing them together a real service will be rendered the profession, as well as much valuable time be saved for considering other and equally important problems.

The history of the coffer-dam process would seem to indicate that engineers of nearly a century ago gave more consideration to the smaller problems than the engineer of to-day, who has apparently passed to the consideration of the larger and of course more interesting ones.

That this is deplorable, is proven by the many cases where money has been wasted in the after effort to make good the mistakes that have become apparent where cheap construction of coffer-dams has been resorted to. The saving in original cost, as between an indefensible method and a defensible one, is often so small as to seem absurd when it has become necessary to make large expenditures to rectify the errors.

Errors of judgment are more easily excusable with regard to foundations than with any other class of construction, but where definite limits can be set, economy will result by keeping as closely as possible within them.

Reference is made in the following pages to the splendid construction of foundations by the Romans, where they could be built outside the water. The Pont du Gard, illustrated in the frontispiece, is the most notable example of this extant. It is interesting also as indicating their knowledge of the better form of piers and methods of arch construction.

Although constructed during the reign of the Emperor Augustus, at the beginning of the Christian era, it is in a remarkable state of preservation, aside from repairs that have been made from time to time.

Probably the earliest recorded examples of the use of coffer-dams which give details of construction are those constructed under the engineers of the Ponts et Chausées.

Those built under Perronet at the bridge of Orleans were large and extensive, and references made to the pile-drivers and the pumps used on the work, serve to illustrate the great amount of attention paid to planning the details of construction.

The same engineer completed the piers of the bridge at Mantes, where the coffer-dams were constructed to inclose both the abutment and the nearest pier within one dam, making the dimensions about 150 feet by 200 feet in the extreme.

Hardly less notable were the coffer-dams at Neuilly, where the interiors were so large that the excavation did not approach near the inside wall of the dam.

All of these were constructed prior to the year 1775, and the details as shown in the elaborate drawings are of much interest to the engineer engaged on similar works.

The coffer-dams constructed about 1825 by Rennie on the new London bridge were the prototypes of those used at Buda Pesth, but were elliptical in form. They were designed with as much care, apparently, as any other feature of the bridge, and from the fact that the water was pumped to twenty-nine feet below low water and the work found tight, the details must have been very carefully executed.

However great the amount of care bestowed, there will be cases undoubtedly where the difficulties cannot be foreseen, and it will become necessary to adopt some of the many expedients cited to overcome them; or they might better be employed from the start, where any suspicion is had that trouble may ensue.

The question as to whether it will be best to use a crib or a sheet-pile coffer-dam will almost always be decided by the character of the bottom, the location, and the character of the foundation to be built. It is advisable, whichever type is selected, to make the size large enough, so that the excavation may be completed without approaching too close to the inside wall of the dam, and so that plenty of room may be had for the laying of the foundation-courses.

The unit stress adopted for timber construction is believed to be as large as will give good results in the majority of cases, both on

account of the possibility of the construction having to undergo more severe usage than is expected, and on account of the grade of timber which is most often made use of for temporary works.

Where it is permissible from the standpoint of true economy, it is believed that steel construction will commend itself for use. In most localities it will not be long until metal construction will be found cheaper than timber for building coffer-dams, and in many places this is already true.

A great mistake is made, in nearly nine cases out of ten, by trying to use old machinery, such as hoisting-engines, pumps, and the like, which are ill adapted to the purposes for which they are intended, on account of lack of capacity, and only too often on account of having outgrown their usefulness.

The engineer would avoid many unpleasant situations by demanding that a proper outfit be provided, and in the end gain the thanks of the contractor for increased profits.

Extended acquaintance with Portland cement is increasing the use of concrete in construction, and this is a great gain for the engineer, as it is not only superior to much stone that is used, but is better adapted to use in difficult situations. It also lends itself more readily to use for ornamental details in pier construction. That truly ornamental piers are not, however, those with needless and frivolous details, has been clearly set forth in the last article. Simplicity and beauty are near relatives. The best locations cannot always be chosen for piers, but careful examination will often be the means by which bad locations may be avoided.

The methods for determining the economic division of a given crossing of a river have not come into general use, probably on account of lack of easy application. The method given is an accurate one and very simple to use, especially if the results are tabulated for a given loading.

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TABLE OF COFFER-DAMS.—

No.	Page.	River and Location.	Current.	Water Head.	Character of Bottom.
1	6	River 200 feet wide, Ohio	None.	12′+	Cemented gravel.
2	6	Clyde at Glasgow	Slight.	9′+	Gravel, sand, mud.
3	8	Estuary or Harbor	Tide.	40′	Sand & gravel over clay.
4	9	Danube at Buda Pesth	Swift.	54′±	Gravel over clay.
5	23	Kanawha near mouth	Swift.	34′−	Gravel over hardpan.
6	25	Ohio near head	Moderate.	20′+	Gravel.
7	25	Western part United States	Moderate.	6′+	Soft.
8	26	St. Lawrence, lower river	Swift.	20′+	Rock.
9	27	Arnprior Bridge	Swift.	21′+	Rock.
10	29	New Mexico, underflow	None.	15′+	Sand.
11	29	Arkansas at Tulsa	Moderate.	7′+	Gravel over rock.
12	29	Western part United States	Moderate.	6′+	Soft.
13	29	Republican in Kansas	Moderate.	6′+	Sandy.
14	29	Western part United States	Moderate.	7′+	Gravel over soapstone.
15	29	Western part United States	Moderate.	6′+	Rock.
16	29	Payette and Weiser, Union Pac.	Moderate.	6′	Soft.
17	31	Mississippi, Fort Madison	Swift.	19′	Soft.
18	31	Schuylkill near Phila., Pa	Moderate.	Deep.	Mud over rock.
19	36	Green River, Wash	Swift.	18′	Gravel and boulders.
20	40	U. S. Canal, Keokuk	None.	12′+	Rock.
21	45	Mississippi, St. Louis	Swift.	22′	Rock.
22	46	Queen's Bridge	Swift.	15′	Rock.
23	47	Harlem Ship Canal	Moderate.	25′	Rock.
24	49	Arthur Kill Bridge	Tide.	28′	Clay over rock.
25	50	Coteau Bridge, C. Pac. Ry	Moderate.	28′	Rock.
26	50	Mystic River, Boston	Tide.	38′	Rock.
27	115	Ann Arbor, Mich., M. C. Ry.	Moderate.	6′+	Gravel.
28	116	Arthur Kill Bridge	Tide.	30′−	Mud and clay.
29	116	Boston Harbor, sewer	Tide.	10′	Sand and gravel.
30	119	Illinois River, La Grange	Moderate.	7′	Sand and mud.
31	119	Kankakee at Momence	Moderate.	6′+	Rock.
32	120	Potomac at Harper's Ferry	Swift.	6′+	Rock.
33	120	Charlestown Bridge, Boston	Tide.	6′+	Soft.
34	121	Fort Monroe, sewer	None.	20′	Soft.
35	123	Seattle, Wash	Tide.	32′+	Soft sand.
36	129	Tennessee at Chattanooga	Swift.	8′+	Gravel over rock.
37	130	Cumberland, Md.	Moderate.	10′+	Sand over hardpan.
38	132	Mississippi, Sandy Lake	Swift.	8′+	Sand.
39	134	Arkansas, Little Rock	Moderate.	6′+	Sand.
40	157	Farnitz, Stettin, Germany	Moderate.	25′+	Clay.
41	159	Coosa, Gadsden, Ala.	Moderate.	10′+	Gravel over rock.
42	134	Schuylkill, P. & R. R. R.	Swift.	8′+	Rock.
43	137	St. Helier Bridge, Jersey, Eng.	Tide.	13′+	Earth over rock.
44	137	Thames at Putney	Moderate.	Deep.	Mud.
45	137	Victoria (B. C.) Docks	Tide.	35′	Rock.
46	138	Kaw at Topeka	Swift.	6′+	Sand.
47	II.8	Firth of Forth	Tide.	15′+	Rock.

SYNOPSIS OF EXAMPLES

Form of Construction.	Inside Dimensions.	Kind of Puddle.	Thickness Puddle.	Remarks.	Page.	No.
Earth bank.	10'×60'?	Clay and gravel.	5'+	No leaks.	6	1
Sheet-piles.	20'×58'?	Clay.	3'		6	2
Sheet-piles.	Large.	Clay, sand, & gravel.	3–6'	Typical.	8	3
Sheet-piles.	72'×136'+	Clay and gravel.	2–5'	Difficult.	9	4
Earth bank.	90'×330'	Clay and gravel.	19'+		23	5
Earth bank.?	200'×600'	Clay and gravel.		Failed.	25	6
Crib.	Medium.	Clay.	3'+		25	7
Crib, single.	24'×43'	Concrete inside.			26	8
Crib, single.	16'×34'	Concrete inside.			27	9
Crib, single.	17'×43'	Clay outside.		Special.	29	10
Crib, single.	Medium.	Clay outside.			29	11
Sheet-piles.	Medium.			Typical.	29	12
Sheet-piles.	Medium.	Clay outside.			29	13
Sheet-piles.	Medium.	Clay outside.			29	14
Sheet-piles.	Medium.	Clay.	Equal depth.		29	15
Box or crib.	12'×36'	None.		On grillage.	29	16
Staves.	36' diam.	None.		On grillage.	31	17
Sheet-piles.	80' diam.	None.		Failed.	31	18
Log crib.	Large.	Clay and gravel.	6'+	Seep. large.	36	19
Canvas on plank.	80' long.	Rotten manure.		Bulkhead.	40	20
Crib, double.	28'×64'	Clay.	3' 0"	Canvas used.	45	21
Box and canvas.	Square.	Clay oustide.		Movable.	46	22
Polygon crib.	47' diam.	Clay.	4' 6"		47	23
Polygon crib.	44' diam.	Clay and gravel.	5' 0"		49	24
Crib, single.	34' diam.	Concrete inside.			50	25
Basket crib.	60' diam.	None.			50	26
Sheet-piles.	13'×44'	Clay and gravel.	2' 8"		115	27
Sheet-piles.	Large.	None.		Two trials.	116	28
Sheet-piles.	12' wide.	Clay.	6–8'		116	29
Sheet-piles.	Medium.	None.			119	30
Sheet-piles.	Medium.	Gravel.		Two trials.	119	31
Sheet-piles.	Medium.	Gravelly clay.			120	32
Sheet-piles.	18'6"×119'	Concrete inside.			120	33
Sheet-piles.	44' diam.	Sand and concrete.	7'+		121	34
Sheet-piles.	30'×56'	None.		Very Tight.	123	35
Sheet-piles.	Large.	Clay.	9' 0"		129	36
Sheet-piles.	15'×50'	None.			130	37
Sheet-piles.	829' long.	Clay.	8'±		132	38
Sheet-piles.	16'×38'	Earth outside.			134	39
Sheet-piles.	23'×55'±	Clay.	2–4'	Removal.	157	40
Sheet-piles.	28'×28'±	Clay.	12'+	Removal.	159	41
Sheet-piles.	16'×42'.	Clay and gravel.	8'+	Movable.	134	42
Sheet-piles.	Medium.	Clay outside.			137	43
Sheet-piles.	Medium.	None.			137	44
Sheet-piles.	500' long.	Clay.	2–7'		137	45
Sheet-piles.	18'×55'	Clay outside.			138	46
Metal.	60' diam.	Concrete seal.			II. 8	47

ENGINEERING AND BUILDING FOUNDATIONS

CHAPTER I

HISTORICAL DEVELOPMENT

THE continued increase in the weight of our bridge superstructures and of the loads they have to carry has led to increased care, to a very gratifying degree, in the preparation of the foundations for bridge piers and abutments.

An old authority very truly states "The most refined elegance of taste as applied in the architectural embellishment of the structure; the most scientific arrangement of the spans and disposition generally of the superior parts of the work; and the most judicious and workmanlike selection and subsequent combination of the whole materials composing the edifice, are evidently secondary to the grand object of producing certain firm and solid bases whereon to carry up to any required height the various pedestals of support for the spans of the bridge."

There is every reason to believe, from the bridges of the Romans still extant and of those of ancient and mediæval times of which there are remains or records, that the foundations were carefully considered.

The most ancient form was likely begun by dumping in loose stones until the surface of the water was reached and the masonry could then be commenced without the necessity for any method of excluding the water. The oldest civilizations were in tropical or semi-tropical countries where the streams are dry beds for many months in the year and suitable foundations were easily made without water to contend with. Where the bottom of the stream was rock, the engineering could be very little improved upon to-day, and even where there was shallow water on rock bottom, the piers were well founded in the shallowest places; the bridge often winding

across the stream in serpentine form, similar to the bridge over the river Karun, at Shuster, Persia. (Fig. 1.)

The arch was developed to such an extent by the Romans and the spans were increased to a length which rendered the construction of piers in the water unnecessary for short bridges, the abutments or skew-backs being without the stream on either bank.

The difficulty of founding piers in midstream was doubtless the controlling cause for the larger spans, such as the one built at Trezzo, over the river Adda, by order of the Duke of Milan, some time prior to the year 1390. The span at low water was 251 feet, the single arch being of granite in two courses. The placing of a middle support was doubtless found to be impracticable and caused the design of an arch which has seldom been equaled or eclipsed. (Fig. 2.)

The construction of roads has ever been the harbinger of civilization, and with the spread of civilization came a demand for the improvement of means of communication. The engineer was called upon to construct better and greater bridges in a permanent manner, which led to the origin and development of the four methods for founding in water that were used in olden times. These may be classified as, first, the method with open caissons; second, the use of piles with a capping of coarse concrete about the tops; third, the use of piles after the manner of the French encaissement; and fourth, the use of coffer-dams. A fifth method might be added, in which the bridge was built on dry land adjacent to the stream, and the river diverted to a new channel afterwards excavated under the completed structure. This is, however, an avoidance rather than a solution, unless the river is to be diverted in the course of its improvement.

The first method, as described in old treatises or accounts, consisted of little more than baskets formed of branches of trees, weighted with stone to sink them, and after sinking filled with loose stone to near low-water level, where the masonry could be commenced. These baskets were similar in construction to the mattresses used in the bank revetment of the Mississippi or the bamboo casings used by the Japanese to hold stones in place on bank protection.

An improvement was effected by using in place of baskets, boxes or small open caissons which were sunk and filled in the same manner, several being used for one pier. This was the method used at Blackfriars bridge and also at Westminster bridge, over the Thames, and has been much used in recent times, the caisson

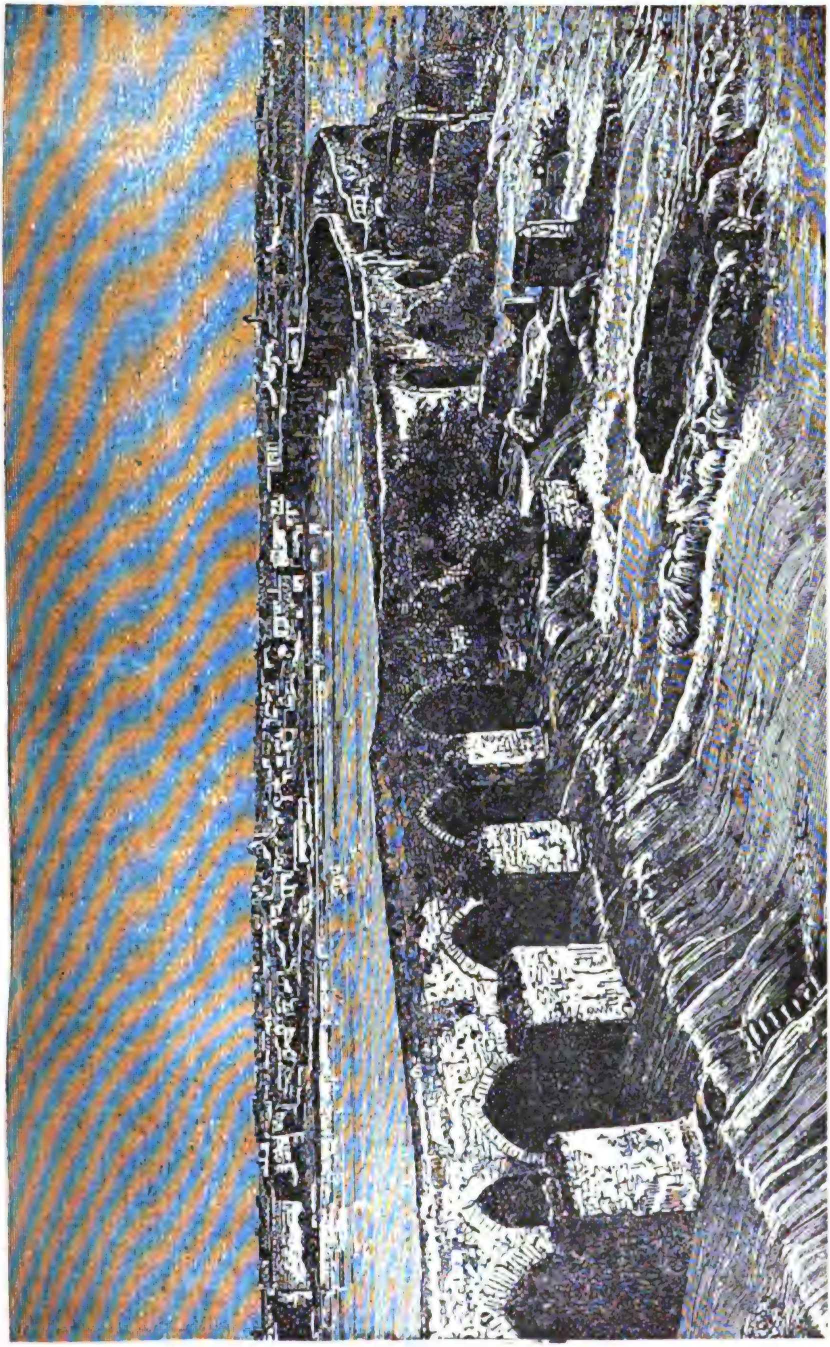


FIG. 1.—BRIDGE AT SHUSTER, PERSIA, OVER THE RIVER KARUN.

being built large and strong enough for the entire pier, which is built up as the caisson sinks.

The second method consisted of driving piles over the area of the foundation until the heads were below low-water level, and spaced at distances apart as required by the nature of the bottom, similar to the methods in vogue to-day. The heads of the piles were not driven to the same level, however, and were incased in a form of coarse concrete such as was used by the Romans, but what is now called beton. This was leveled up and on it was laid the stone for the footing course of the pier.

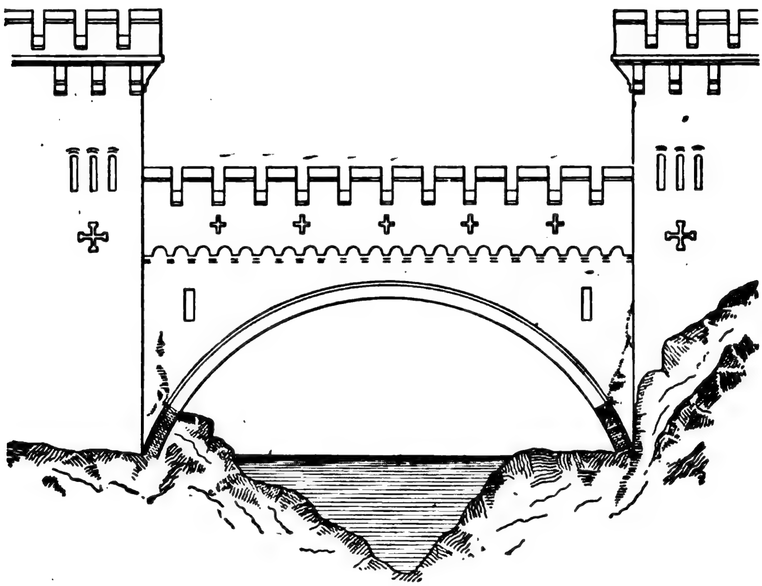


FIG. 2.—BRIDGE OVER THE ADDA AT TREZZO, MILAN, A PROBABLE RESTORATION. (The shaded portion of arch rings is all that remains.)

The third method of encaissement was probably an improvement of the dumping in of loose stone on which to place the pier, and consisted in inclosing the space for the pier with sheet-piling, after which the loose material was removed from the bottom as much as possible and the stone dumped inside until nearly up to low water, at which time the pier could be begun.

These last two methods doubtless met with much favor, owing to the familiarity with pile-driving, in which the Romans especially were proficient. Cæsar's bridge over the Rhine was built entirely on piles, and in a view of it after the old print in the Museum de St.

Germaine, is pictured a pile-driver on a float in position for driving. (Fig. 3.)

This third method was the early type of the crib, which has been such a factor in the building of the earlier foundations over our American rivers. Crossed timbers laid up crib fashion with rectangular openings or cells between the timbers were sunk and filled with broken stone on which to build the pier.

These methods were all deficient in affording no means of seeing or making a careful examination of the bottom on which the foundation was to be placed, and with the advent of more permanent structures of greater magnitude the coffer-dam came into use. This

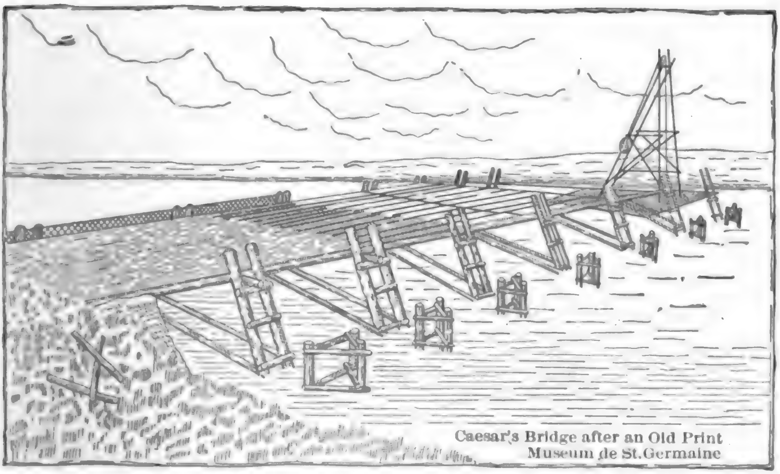


FIG. 3.—CÆSAR'S BRIDGE OVER THE RHINE.

allowed the bottom to be freed from water, and after a careful examination and preparation of the foundation, the work could proceed in the dry until above water-level.

The pneumatic caisson is now in general use for all foundations that must go to any considerable depth below the water and has even been used in some instances where the depth was slight, but where for various reasons it was deemed expedient to use compressed-air caissons. Recent expressions from some engineers of high standing would indicate that they do not consider it good practice to use coffer-dams in any case, one making the statement that he had not used a coffer-dam for thirty years, while another seemed to think it a matter to be left to the pleasure of the contractor. That the use of this method has gotten into disfavor seems to be

beyond question and it will be the purpose of the succeeding pages to learn to some extent why this is so, but mainly to show from successful examples how to proceed, that success instead of failure may result. Any attempt to account for the origin of the coffer-dam process would be futile, inasmuch as the savage, wishing to free a space from water, doubtless banked up earth about the area and, scooping out the water with his hands, laid the ground bare for inspection. From so simple a beginning, the first method likely to occur to a mind capable of reasoning, can readily be imagined the course of development of coffer-dams.

The most simple form in use at the present time, where the water is quiet, is shown in Fig. 4, and consists principally of a bank of earth which is made thick enough to be nearly or quite impervious to water, the earth being prevented from caving into the excavation by piles supporting a timber casing. Some of the recorded examples of the early use of this process are of interest in illustrating the care which has been bestowed upon their construction in important works and will call attention to that incessant care which is necessary to success in any work of this character.

Robert Stevenson, the great English engineer, thought it not beneath his dignity to give full instructions as to the construction of the coffer-dams for the Hutcheson bridge over the Clyde at Glasgow. The bridge consisted of five arch spans, the total length between the abutments being 404 feet and the width 38 feet. The four piers were from 11 to 12 feet in thickness, being designed to take up the arch-thrust, and 48 feet in length at the footing. The specifications written at Edinburgh in April, 1828, are so explicit that they will be quoted in full on this point: "It having been ascertained by boring and mining that the subsoils of the bed of the river consist of gravel, sand, and mud to the depth of 27 feet and upwards, it becomes necessary to prepare foundations of pile-work for the bridge; and, therefore, to insure the proper and safe execution of the works, coffer-dams are to be constructed around each of the foundation-pits of the two abutments and four piers of such dimensions as to afford ample space for driving piles, fixing wale-pieces, laying platforms, pumping water, and setting the masonry; and likewise for the construction of an inner or double coffer-dam should this ultimately be found necessary. The framework of the coffer-dams is to consist of not less than two rows of standard or gage- and sheeting-piles, kept at not less than 3 feet apart for the thickness of a puddle-wall or dyke, which space is to be dredged to a depth of not less than 9 feet under the level of the summer water-mark above de-

scribed, before the sheeting-piles are driven. The gage or standard piles are to measure not less than 24 feet in length and 10 inches square. They are to be placed 3 yards apart and driven perpendicularly into the bed of the river to the depth of 16 feet under the level of the summer water-mark, thereby leaving 8 feet of

FIG 4.—A PRIMITIVE SOLUTION.

their length above that mark. Runners or wale-pieces of timber 9 inches square are then to be fitted on both sides of each row of gage-piles, to which they are to be fixed with two screw-bolts of not less than 1 inch in diameter, passing through each of the gage-piles. One set of these inside and outside wale-pieces is to be placed at or below the level of summer water-mark, and the other

set within 1 foot of the top of each row of said piles, the whole to be fixed with screw-bolts in the manner above described. The wale-pieces are to be 4½ inches apart in order to receive and guide the sheeting-piles. This is to be effected by notching the wale-pieces into the gage-piles. The sheeting-piles are to be 21 feet in length, 4½ inches in thickness, and not exceeding 9 inches in breadth. They are to be closely driven, edge to edge, along the space left between the walings, and each compartment of the sheeting between the gage-piles is to be tightened with a key-pile. The coffer-dam frames are to be properly connected with stretchers and braces before commencing the interior excavation. Each coffer-dam is to be provided with a draw-sluice, 14 inches square in the void, with a corresponding conduit passing through the puddle-dyke at the level of summer water-mark. To render the coffer-dams water-tight the whole excavated space between the two rows of piling is to be carefully cleared of gravel, sand, or other matters, to the specified depth, and clay well punned or puddled is then to be filled in and carried up to the level of the top of the sheeting-piles. But if it shall, notwithstanding, be found that the single tiers of coffer-dam do not keep the foundation-pits sufficiently free of water for building operations, the water must either be pumped out and kept perfectly under by steam or other power, or else excluded by the construction of a second tier of coffer-dam similar in construction to the first. For the foundation-pits of the two abutment piers on either side of the river it is not expected that more will be required on the landward side for keeping up the stuff than a single row of gage- and sheeting-piles; but if the engineer shall find other works necessary upon opening the ground they must be executed by the contractor and shall be paid for agreeably to the contract schedule of prices for the regulation of extra and short works. The stuff within the coffer-dams is to be excavated to the depth of 10 feet under the level of summer water-mark for each of the piers and 8 feet for each of the abutments."

The present practice of leaving all this to a contractor, whose idea is too often to sacrifice everything to cheapness, appears in very unfavorable contrast to this careful description.

An article on founding by means of coffer-dams, published in 1843, gives directions for placing a coffer-dam in 40 feet of tide-water; and although the engineer of to-day might use some other method for such a depth, an illustration (Fig. 5) and short description of it are given, as ideas may be gained for application to ordinary works.

The water was assumed at 10 feet deep for low tide, 28 feet at high tide, with 12 feet of sand and gravel to be removed to expose the clay on which the pier was to rest. Four rows of piles were to be driven around the area, the outer row to within 1 foot of low water, the two rows in the middle to within 3 feet of high water, the inner row to 11 feet above low water, and all to be down 5 feet into the clay. The outer row of piles to be 6″×12″, the two rows in the middle 12″×12″, and the inner row 8″×12″; all driven close together and to have waling-pieces, braces, and brace rods as shown in cross-section. The rows to be 6 feet apart and to be filled in between with a puddle of clay mixed with sand and gravel.

The report of W. Tierney Clark, the engineer of the Buda Pesth suspension bridge, gives an account of what are probably the largest

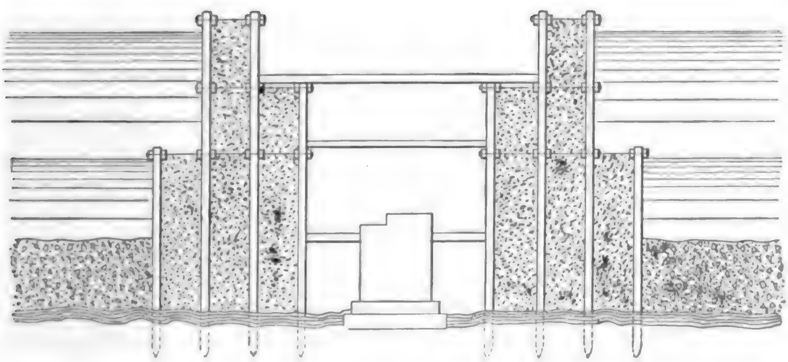


Fig. 5.—Coffer-dam in Tide-water.

bridge coffer-dams ever constructed. Some other method would now be used for such a location, but this fact will not detract from the lessons that may be drawn from them.

The Danube was crossed at Buda Pesth previous to the year 1837 by means of a bridge of boats which had to be taken up during the winter and the passage made by ferry or on the ice, so that for six months of each year there was great risk in crossing and frequent loss of life. The building of a permanent bridge was brought about through the efforts of the Count Szechenyi, who, as a member of a committee, proceeded to England in 1832 and after a careful investigation of existing works decided upon the construction of a suspension bridge. The greatest question for solution was the founding of the two towers in a river like the Danube, where the ice throughout the long winter wrought havoc with everything in reach. The ice in the river in February, 1838, was from 6 to 10 feet thick

near the site of the proposed piers. On March 9, a movement occurred across the whole river and for a length of 350 yards, the whole moving in a solid mass. On March 13 it moved again 400 yards and three hours later a general breaking began. The ice piled up on the shoals, causing a sudden rise to 29 feet 5 inches above zero, and while it was at this height for only a few hours, it is recorded that a great part of Buda and two-thirds of Pesth were destroyed and many lives lost.

The extraordinary design of the coffer-dams can the more readily be understood after this description, it being doubted by many

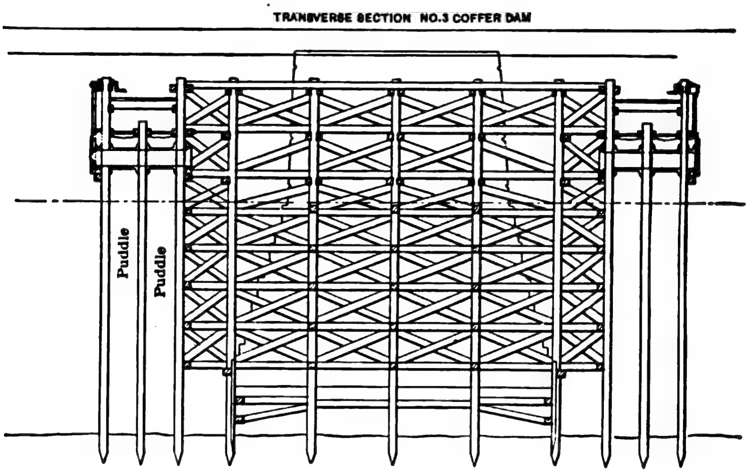


FIG. 6.—BUDA PESTH SUSPENSION BRIDGE.

persons at that time whether piers could be placed in the river by any means. (Fig. 6.)

The drawings reproduced are of coffer-dam No. 3, which was about 72 feet in width and about 136 feet in length inside the puddle walls, there being two puddle chambers, each 5 feet in width. From a point about 13 feet above the clay on which the tower was to rest, was an inside wall of sheet-piling, this space being nearly filled, after excavating, with a bed of concrete. The piling of each row, from 40 to 80 feet in length, was all carefully sized to 15 inches square, shod with iron and driven close together, penetrating 20 feet below the bed of the stream or 40 feet below the zero level. The framing of the ice-breaker and the bracing within the dam was of enormous strength. The number of piles driven in the four coffer-dams reached the enormous total of 5,224, and of the 1,227 driven in dam No. 3, $16\frac{1}{4}$ per cent. were drawn and redriven. These piles

and the timber were obtained from the forests of Bavaria and Upper Austria. (Fig. 7.)

The first pile on dam No. 3 was set on April 8, 1842, but owing to the difficulties encountered it was not finished until three years later—April 4, 1845. From six to seven days were occupied at the first in driving a pile to a depth of 5 or 6 feet into the clay, but as the work progressed the difficulty increased, the operation of driving one pile consuming from twelve to fourteen days, many piles breaking short off so they could not be withdrawn, and the gravel was dredged out from behind and a second row driven. The report further describes the difficulty of the work: "The dredging for the No. 3 dam was carried on to the average depth of 44 feet from the top of the outer row of piles, leaving about 10 feet of gravel to drive through, and extra piles were driven where the gravel found its way between the piles, as well as where it was known the piles were not driven to the proper depth, or were broken or otherwise injured. As the gravel was dredged out to the above depth, the inner and middle row of piles were driven, and a great part of them got down as was supposed to the requisite depth. The work was carried on in the above manner until the 7th of November, when from the appearance of several piles which were pulled up, and from other causes, it became apparent that the outer row was in a much worse state than had been expected, and it was almost a matter of certainty that those piles which had taken ten or twelve days to get down were not driven to the proper depth by at least 3 or 4 feet, having upset or lost their points to that extent. There was likewise every reason to believe that many of them were broken or dangerously crippled. Added to this the Danube was rising, and at the late time of the year, with winter rapidly approaching, the general appearance of the dam was anything but satisfactory. Upon mature consideration the only course appeared to be to drive a much greater number of piles than was at first calculated upon, and another complete row of piles was driven all around at intervals of 15 inches apart, and in some cases double and triple piles were driven during the progress of the dredging. At the commencement of the driving a few were got down to the depth of 57 or 58 feet, being from 3 to 4 feet in the clay; but as the gravel began to get compressed many of them would not penetrate more than 54 or 55 feet, the sharp, angular gravel overlying the clay appearing to be compressed into a substance as hard as rock."

The puddle used was clay mixed with about one-third clean gravel, it having been found to set quite solid, from experiments made by

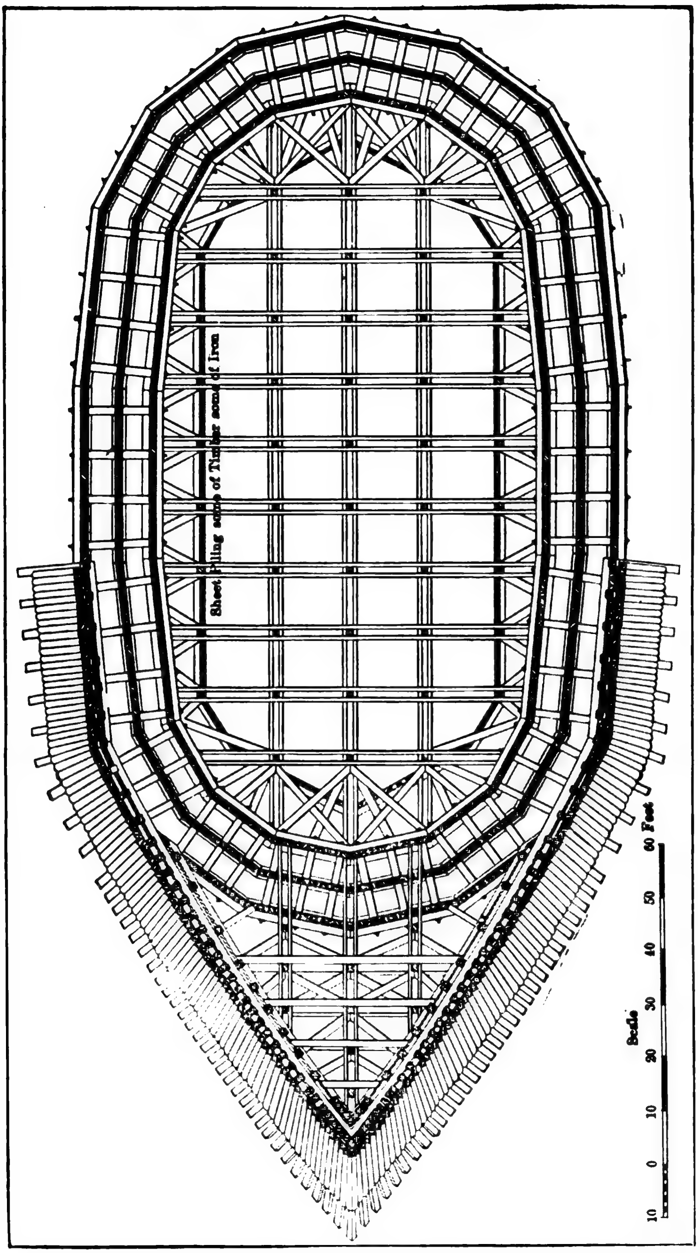


FIG. 7.—BUDA PESTH SUSPENSION BRIDGE, PLAN OF COFFER-DAM NO. 3.

sinking specimens in the Danube. When leaks occurred they were closed by driving square timbers down 30 or 40 feet into the puddle to pack it, or by driving new piles to close the cracks, and in some cases by driving sheet-piling.

Experiences of this nature led to the disuse of coffer-dams for foundations to such depths, but a very small percentage of the care exercised and the persistence shown in this work would lead to greater success on ordinary foundations.

The class of work to which coffer-dams may still be applied will be shown in the succeeding pages and the examples from actual practice will show in some measure the care that must be exercised in the first construction to prevent failure, and the expedients adopted to overcome unavoidable accidents.

The historical features of foundations are of sufficient importance to add to the foregoing pages a résumé of the earliest uses of the various methods described in the subsequent chapters.

Vitruvius is probably the first writer on the subject and he gives instructions as to the methods for making the foundation which can be little improved upon to-day.

"Foundations should be carried down to solid bottom, if such can be found, and they should be built thereon of such thickness as may be necessary for the proper support of that part of the wall standing above the natural level of the ground. They should be of the soundest workmanship, and materials of greater thickness than the walls above. . . .

"The intervals between the foundations brought up under the columns should be either rammed down hard, or arched, so as to prevent the foundation piers from swerving. If solid ground cannot be come to, and the ground be loose or marshy, the place must be excavated, cleared, and either alder, olive, or oak piles, previously charred, must be driven with a machine as close to each other as possible and the intervals between the piles filled with ashes. The heaviest foundations may be laid on such a base."

Thus we find that the use of piles was of a very early date, and rubble mounds for the base of piers, similar to those already mentioned for the bridges of Persia, of very ancient origin. Earthen dams for excluding the water are known to date back to the seventeenth century.

Piles surrounded by rubble stone, in place of the ashes mentioned by Vitruvius, were used at the Notre Dame bridge in Paris between 1500 and 1507. Piles supporting a platform on which to build piers were used at Blois in the year 1716.

Régemortes made use of an apron in preparing a foundation at Moulins in 1750. As early as 1750 caissons were sunk for the piers of the old bridge of Westminster, after the bed of the river had first been dredged, a similar process having been used in 1686, or 64 years earlier, at the Tuileries bridge.

The cutting off of piles under water was accomplished in 1756, by the use of a saw invented by Des Essarts, and this method was used on a great many bridges for the next hundred years.

The discovery in the year 1818 of the properties of hydraulic mortars, by Vicat, made possible the forming of a foundation by depositing concrete inside of sheeting and also the use at Paris by Beaudemoulin for the first time of a bottomless frame or crib, with concrete at the bottom. This method was improved upon by Poirel about 1840, by adding a canvas bottom to the crib, or caisson, to deposit concrete *in situ*.

The earliest account we have of the use of compressed air is given by John Taisner of Hainault, born in 1509, who went to Toledo with the Emperor Charles V, where he saw two Greeks let themselves down under water in an "inverted caldroun" with a light, and return to the surface without getting wet. Lorini also describes the use of the diving bell or the progenitor of the pneumatic caisson, and the first account of its use in England is given by Dr. Halley early in the eighteenth century.

The diving bell used by Smeaton at the Hexham bridge in 1778, is really the first use of compressed air on bridge foundations of which we have an authentic account. His account of it in a letter of instructions gives the details of construction (Fig. 8) and method of using it. The details of the air pump are shown in Fig. 9.

Smeaton employed the diving bell as early as 1778, in the construction of the bridge at Hexham; in a letter which he addressed to Mr. Pickernell on the subject, we have the method he adopted fully explained; he says: "If the cases would have enabled us to reduce the water so low, as to be even with the very bottoms of the caissons of each pier, I take it for granted, you would have thought it no difficulty with broken rubble, beton, stones, and short blocks of wood, cut a little wedgeways, to have crammed and wedged up the cavity washed under the wooden bottoms, so as to have been equally resisting, and capable of bearing a weight with the original gravel, and particularly when this new body of matter is supported, and even jammed tighter into its place by filling up the vacancy between the pier and the base, a little above the wooden bottom, with rubble, and then driving it tight down by a set with the ram.

It therefore now remains that I describe, and make you master of a piece of machinery, that will put you nearly into the same condition, as if the water could have been reduced to the caisson's bottoms as before mentioned; and this is by means of an air-chest, or diving vessel, which being let down, will exclude the water down to the very bottom of the river if you please, and therefore as low as the under side of the wooden bottom, which in the present case is as low as will be necessary or useful, and the chest or vessel being large enough to give liberty for a man to work therein; being furnished with a pair of boots, he will at mid leg deep in water, do his business with almost as much facility as if the water were pumped out to the same level. The principal part of this machine will consist of a strong chest (Fig. 8), suppose 3 feet 6 inches in length, about 4½ feet deep or height and as wide as to give free leave for its going down between the cases and the piers, which I suppose will be about 2 feet wide inside measure, as the other measures are also supposed to be. Now you know very well that if you push a drinking glass, or any other similar vessel, with its mouth downwards into the water, that it will exclude the water, leaving the vessel full of air, as it was before it was thrust into the water; in like manner, if this chest, being loaded with a sufficient weight, be let down into the water, mouth downwards, the air will exclude the water to the bottom skirt of the chest, and if let down, so as to rest upon the bottom of the river, a man may stand therein, and do any kind of business, the same as he could do in the same space in the open air. But to continue this for any length of time two things are obviously necessary, and those are light and a circulation of fresh air. The former might on occasion be supplied by a candle, but here we may have the advantage of day-light by putting two or three strong round panes

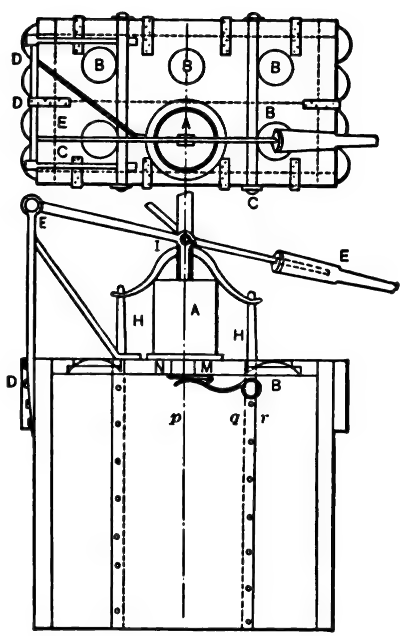


FIG. 8.—SMEATON'S DIVING BELL.

of glass into the bottom of the chest, which will in its inverted situation be the top; a sufficiency of light will enter, this top of the chest being supposed above water. Respecting air, you will conceive that any quantity might be forced in by a strong pair of bellows; but those made of leather would be cumbersome and unhandy; I therefore substitute a kind of foreign air-pump (Fig. 9), made of thin hammered copper, that will throw in a gallon at a stroke, which will not only continually refresh the workmen within, but whatever air escapes through the joints or pores of the air-chest will be replenished, and the overplus go out at the bottom or skirt of the chest, and boil up on the outside.

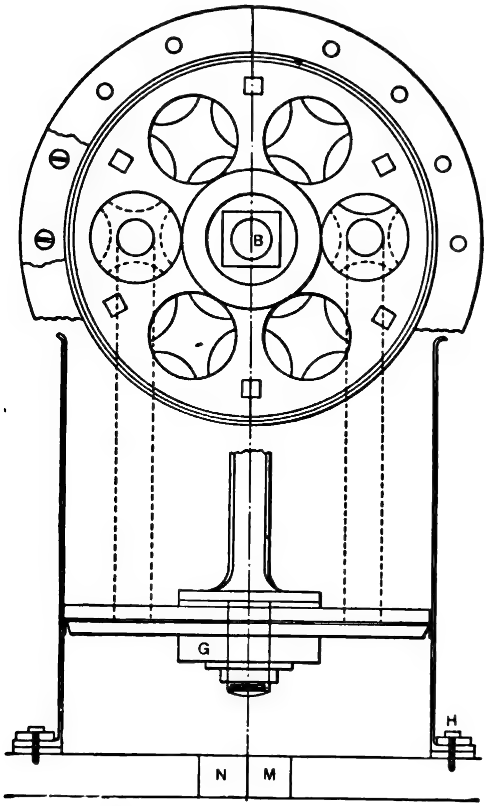


FIG. 9.—SMEATON'S AIR PUMP.

"The quantity of weight that will sink it, mouth downwards, will be the same as placed therein (bottom downward) would sink it the same depth; and as this chest I propose to be suspended by a tackle, and to go down by its own weight, I compute that it will take sixteen pigs of lead to sink it to the bottom of the river, and keep it steady. I propose that the lead may be as much out of the way as possible, to place them upon the ends of the chest, endways upward, that is four in a row below and four above, and the same at the other end, making in the whole 16 pigs, which are to be fastened on with screws, either by cleats screwed on, or punching a hole through each end of each pig. At one end of the chest there is to be a board, fixed across for the man to sit upon, and a cleat nailed to each side to set each of his feet upon, so that while the machine is

being lowered or hoisted, he is totally dry, and when let down enough, he stands upon the bottom of the river, without any more water than the height between the skirt of the chest and the bottom of the river, which may be more or less as is found convenient, I suppose never more than a foot deep, because wherever the ground is taken out more than 1 foot below the underside of the caisson's bottom, I would propose to fill it up with rubble previously to that height or depth, nor can it be of use to let down the skirt of the chest much below the caisson's bottom, because the side of the chest will then diminish the room you will have to get the matter from underpinning the caisson's bottom. The foregoing will, I believe, be sufficient for explaining the general principles and outlines of the method I mean to pursue in underpinning, and resupplying what is underwashed from the bases of the piers, and which I dare say you will now see to be entirely practicable; what you are therefore immediately to put in hand is the air-chest, of or about the inside dimensions before mentioned; I believe the two flat sides will do very well, if of good red wood deal, shot clean of sap, the two ends and bottom, (or in use its top); it would be well if they could be got of single planks of elm, beech, or plane trees, as they would hold the nails better: I fancy $1\frac{1}{2}$ or $1\frac{3}{4}$ inches thick for the sides, $2\frac{1}{4}$ or $2\frac{1}{2}$ for the ends and bottom, will be sufficient; they should be well jointed, and put together with white lead and oil, as the effort will not be of the water to enter, but of the air to escape from within. Were I with you, when it is put in use, I should be the first to go down into it, as there is no more danger (all your tackle being firmly fixed) than being let down into a coal-pit by a rope: and if it shall happen that all your masons are too fine fingered, I fancy a couple of colliers to take turn and turn will find it a very comfortable job. A particular encouragement, must, however, I expect be given. I will give you more particular directions in my next: as to the air pump, all that will be wanted from the coppersmith will be a cylindrical pipe of copper, 10 inches diameter, and 12 inches high, wired at top, and a flanch at bottom of about $1\frac{1}{4}$ inches broad, by which it is screwed down before the top of the air-chest; the copper to be about the thickness of a halfpenny; if you have no neat-handed coppersmith that can hammer it straight and smooth inside, it may on occasion be made of strong tin."

The references to the letters on Figs. 8 and 9 are: *A*, the air pump; *B*, skylights 6 inches in diameter, to be made of window glass knobs, if plate glass cannot be had; *C*, clamp plates of iron, to hold the sides and top firmly together; *D, D*, pigs of lead, end

upwards; *E, E,* the lever for working the pump; *G, G,* the axis and brace for steadying the lever; *H, H,* two bows for hoisting the chest; *I,* a strong iron bow to hook the tackle to; *M, N,* the opening from the pump to the air chest; *o, p,* the valve, *o* being the leather and *p* the wood; *q r,* the spring to shut it, having just strength enough to shut the valve.

The following account of the improvement of the diving bell by Sir John Rennie is taken from Smiles' "Lives of the Engineers."

"Whilst occupied on the works of the Ramsgate Harbour of which he was appointed engineer in 1807, Mr. Rennie made use of the diving bell in a manner at once novel and ingenious. It will be remembered that Smeaton had employed this machine in the operations connected with the building of the harbour; but his apparatus being wood, was exceedingly clumsy, and very limited in its uses. In that state Mr. Rennie found it, when he was employed to carry on the extensive repairs of 1813. The east pier-head was gradually giving way and falling into the sea at its most advanced and important point. No time was to be lost in setting about its repair; but from the peculiarly exposed and difficult nature of the situation, this was no easy matter. The depth at the pier-head was from 10 to 16 feet at low water of spring tides; besides, there was a rise of 15 feet at spring and 10 feet at neap tides, with a strong current of from two to three knots an hour setting past it both on the flood and at the ebb. The work was also frequently exposed to a heavy sea, as well as to the risk of vessels striking against it on entering or leaving the harbour.

"Mr. Rennie's first intention was to surround the pier-head by a dam; but the water was too deep and the situation too exposed to admit of this expedient. He then bethought him of employing the diving-bell; but in its then state he found it very little use. No other mode of action, however, presenting itself, he turned his attention to its improvement as the only means of getting down to the work, the necessity for repairing which had become more urgent than ever. Without loss of time he proceeded to design and construct a bell of cast iron, about 6 feet in height, $4\frac{1}{2}$ feet wide, and 6 feet long, having one end rather thicker and heavier than the other, that it might sink lower, and thus enable the exhausted or breathed air more rapidly to escape.

"At the top of the bell, eight solid bull's-eyes of cast glass were fixed, well secured and made water-tight by means of leathern and copper collars covered with white lead, and firmly secured by copper screw bolts. To the top of the inside were attached two

strong chains for the purpose of fastening to them any materials that might be required for the work, and flanges were cast along the sides of the bell, on which two seats were placed, with foot-boards, for the use of the men while working. In the centre of the top was a circular hole, to which a brass-screwed lining was firmly fixed, and into this a brass nozzle was screwed, having a leathern water-tight hose fastened to it, 2½ inches in diameter. The hose was in lengths of about 8 feet, with brass-screwed nozzles at each

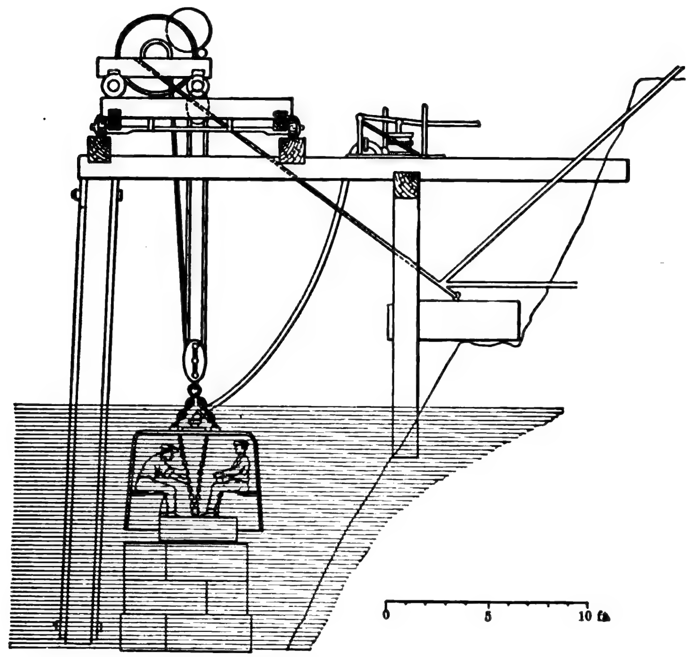


FIG. 10.—RENNIE'S DIVING BELL.

end, so that it could be lengthened or shortened at pleasure, according to the depth of water at which the men in the bell were working.

"For the purpose of duly supplying the machine with air, a double air-pump was provided, which was worked by a sufficient number of men. The air-pump was connected with the hose referred to, and was either placed on the platform above or in a boat which constantly attended the bell while under water. Two stout wrought-iron rings were fixed on the top of the machine to which ropes or chains were attached for the purpose of lowering or raising it. The whole weighed about five tons; and it was attached to a circular framework of timber, strengthened by iron, erected over where the

intended circular pier-head was to be built, and so fixed to a pivot near the centre of the work that it was enabled easily to traverse its outer limits.

"On the top of the framework was a truck, made to move backwards or forwards by means of a rack on the frame, and a corresponding wheel provided with teeth, worked by a handle and pinion. On the truck were placed two powerful double-purchase crabs or windlasses, one for working the diving-bell suspended from it, and the other for lowering stone blocks or other materials required for carrying on the operations at the bottom of the sea. By these ingenious expedients the building apparatus was so contrived as to move all round the new work backwards and forwards, upwards and downwards, so that every part of the wall could be approached and handled by the workmen, no matter at what depth; while the engineer stationed on the pier-head above could at any time ascertain, without descending, whether the builders were proceeding in the right direction, as well as the precise place at which they were at work.

"Everything being in readiness for commencing operations, the divers entered the bell and were cautiously lowered to the place at which the building was to proceed. A code of signals were established by which the workmen could indicate, by striking the side of the bell a certain number of strokes with a hammer, whether they wished it to be moved upward, downward, or horizontally; and also to signal for the descent of materials of any kind. By this means they were enabled, with the assistance of the workmen above, to raise and lower, and place in their proper bed, stones of the heaviest description; and by repeating the process from day to day and from week to week the work was accomplished with as much exactness, and almost as much expedition, under water, as though it had been carried on above ground.

"Thus the entire repairs were completed by the 9th of July, 1814; and to commemorate the ingenuity and skill with which Mr. Rennie had overcome the extraordinary difficulties of the undertaking, the trustees of the harbour caused a memorial stone to be fixed in the centre of the new pier-head, bearing a bronze plate, on which were briefly recorded the facts above referred to, and acknowledging the obligation of the trustees to their engineer. They also presented him at a public entertainment with a handsome piece of plate in commemoration of the successful completion of the work. The diving-bell, as thus improved by Mr. Rennie, has since been extensively employed in similar works; and although detached divers,

with apparatus attached to them, are made use of in deep-sea works, the simplicity, economy, and expeditiousness of the plan invented by Mr. Rennie, and afterwards improved by himself, continue to recommend it for adoption in all undertakings of a similar character."

The first use of compressed-air caissons was by M. Triger from 1839 to 1841 at the Chalons coal mine, and this was rapidly improved upon for foundation work, until we have the modern compressed-air caisson.

Dr. Potts brought out his vacuum process in 1845, in which the air is exhausted from the caisson, and the external air pressure utilized for the weight to sink it; this process, however, is seldom if ever used at the present time.

One of the earliest cases of the use of the pneumatic process for sinking bridge piers in America was at Omaha over the Missouri River in the year 1869, under the direction of Gen. Wm. Sooy Smith. The piers of the St. Louis Eads bridge and of the East River Roebling bridge were sunk by this process between 1870 and 1873.

"In every man's mind, some images, words, and facts remain, without effort on his part to imprint them, which others forget, and afterwards these illustrate to him important laws."

CHAPTER II

CONSTRUCTION AND PRACTICE—CRIB COFFER-DAMS

The exact definition of the term coffer-dam—" a water-tight inclosure, from which the water is pumped to expose the bottom and permit the laying of foundations "—is the class of structure which is to be considered, although in the construction of them cribs or caissons may be employed and utilized; the essential purpose being to form an inclosure as nearly water-tight as possible in order that the expenditure of power for pumping out the water may be of small amount.

The attainment of this when the water is shallow and has little current we have seen to be easily accomplished by means of a bank of clay or clayey gravel.

This form may also be employed in still water up to about 4 feet in depth by the addition of sheet-piling or a casing supported by ordinary piles to prevent the embankment from caving into the excavation. Where the bottom is of soft mud or porous material over a solid clay or gravel, as much as possible of the porous material should be removed before forming the embankment, thus preventing leakage underneath. In very shallow water this can be accomplished by shoveling and with large hoes or scoops, but with several feet of water to contend with, some form of dredge or scraper must be employed. A very convenient form of scraper used by M. L. Byers on the Cinti. & Mus. Valley Railway is described in Vol. 31 of the " Transactions of the American Society of Civil Engineers," and consists of old boiler-iron, strengthened by three ribs of light iron rail as shown in Fig. 11. This was operated by a double-drum 20-horse-power Mundy hoisting-engine, with the towing-line running directly from one drum to the scraper and the back line from the other drum over a sheave to the front of the scraper. The excavating averaged about 45 yards of material each day during twelve days' work. The weight of the device was about one thousand pounds.

Where the material is very soft, a hand-dredge, called a spoon, will accomplish the work at about the same cost as excavating on dry land. The spoon usually consists of a long pole, having a cutting-ring fastened at one end, and to this ring is attached a canvas bag to contain the excavated material. The ring is hung from a derrick with a set of falls, being guided with the pole, as it is dragged forward by the derrick through the material to be excavated.

Excavating may be done on all the larger rivers by employing the sand or gravel diggers which are almost always to be found, the dredging being accomplished by means of a series of buckets on a

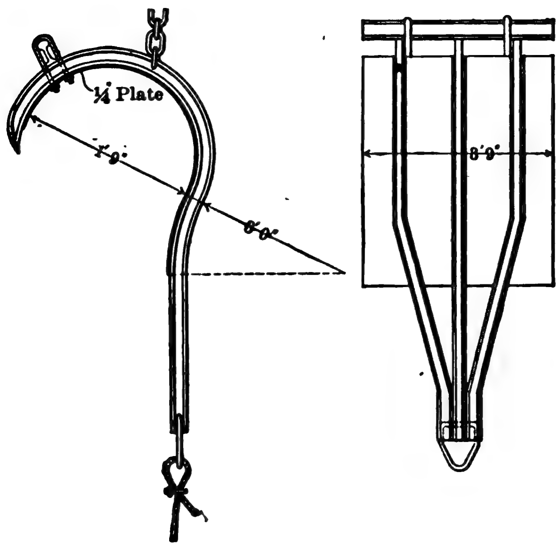


FIG. 11.—SCRAPER DREDGE.

belt or on chains operated through a well in the bottom of a barge. Dredging by machinery on a large scale will be considered later on in some detail.

The method of embankment is sometimes employed for greater depths than 4 feet and in some instances successfully.

The Chanoine dams on the Great Kanawha River required substantial foundations beneath the water, and to accomplish this Addison M. Scott, the resident engineer, employed log cribs about the spaces, with earth banked up on the outside. This work is described in the report of the Chief of Engineers for 1896, the metal work for dams Nos. 9, 10 and 11 being constructed under direction of the author as Chief Engineer of the fabricating company.

The site of the navigation pass of dam No. 11, including the center pier, required a coffer-dam 90 feet wide and 330 feet long inside. (Fig. 12.) This area, including the necessary room for the cribs, was dredged out to hard-pan from 20 to 24 feet below low water. The log cribs, which contained about 84,000 lineal feet of logs, were sunk in sections 19 feet wide and 20 feet long. They were sheathed up to about 3 feet above low water, with sheet-piling in three layers, on the Wakefield system. The driving was accom-

FIG. 12.—COFFER-DAM AT DAM NO. 11, GREAT KANAWHA RIVER.

plished by attaching an eighty-pound weight to an Ingersoll-Sergeant drill run by steam and utilizing the reciprocating motion by attaching the drill with clamps to the tops of the sheathing, following it down as it was driven, after the manner of the Nasmyth steam pile-hammer. This tool, which is one of the most ingenious ever devised for the purpose, was arranged by the contractor's engineer, S. H. Reynolds, and was a complete success.

The tops of the cribs were 10 feet above low water, and the bottoms rested on the hardpan, making a total height of from 30 to 34 feet.

The cribs were filled with sand and gravel that had been dredged out, but the outside was banked up with selected clay and dredged material, which was protected by a layer of riprap up to about low water.

When the coffer-dam was first pumped out several leaks were developed, but after one week in perfecting the details the pumps were started regularly and no serious trouble was had afterward. This is only one of a series of coffer-dams which have been constructed on the several dams in this river, and owing to the care exercised good results were obtained uniformly.

The construction of a similar piece of work on the Ohio River was begun by Major R. L. Hoxie, corps of engineers, and is described in the report of 1895: "It was originally planned to inclose the site of the dam and lock within a coffer-dam, and work was commenced upon that basis. But on attempting to pump out the inclosure it was found that water came in in large quantities, not only under the dam but from springs in the bottom, and all attempts to close these by dumping clay and gravel were a failure. The area inclosed by the dam was about 600 by 200 feet or about 3 acres of river bottom. The deposit of sand and gravel overlying the rock was about 35 feet thick, the rock being 45 feet below the water-level, while the plans required an excavation 20 feet deep below this water-surface. The bottom deposit had been worked over for years by sand-diggers, who threw back the large stones and coarse gravel after removing the fine sand, this work resulting in a very permeable bottom, with possible channels of comparatively large dimensions extending to unknown distances beyond the limits of the coffer-dams."

This is perhaps the most frequent source of failure of a well-constructed coffer-dam and should always be guarded against by removing as much of the porous material as possible, by dredging, before the construction of the coffer-dam is begun.

Cribs are very easy to construct, usually very substantial, and easy to make use of by floating to position and then sinking in place. A very simple form that has been used on the Chicago, Burlington & Quincy Railroad is described by E. J. Blake, chief engineer. Where the water is shallow they have been built in the form shown (Fig. 13), of fence boards spiked one piece on another; with deeper water they are made of heavier timber, $2'' \times 8''$ or $2'' \times 10''$. They are built on the water and are tied across at intervals by pieces spiked through the wall, which pieces should be carefully fitted to prevent leakage. In some cases where the bottom is soft, instead of dredg-

ing, a bottom is added to the crib to prevent the filling from squeezing its way out from under the edge.

When the crib has reached bottom, being sunk by weighting it down if necessary, the chambers are filled with clay puddle and clay is banked up around the outside to prevent water running under. The crib is made large enough so that the excavation will leave an easy slope to the inner edge of the timber work. This form can be made to conform readily to the contour of the bottom by starting the layers of timber at different elevations. No leakage

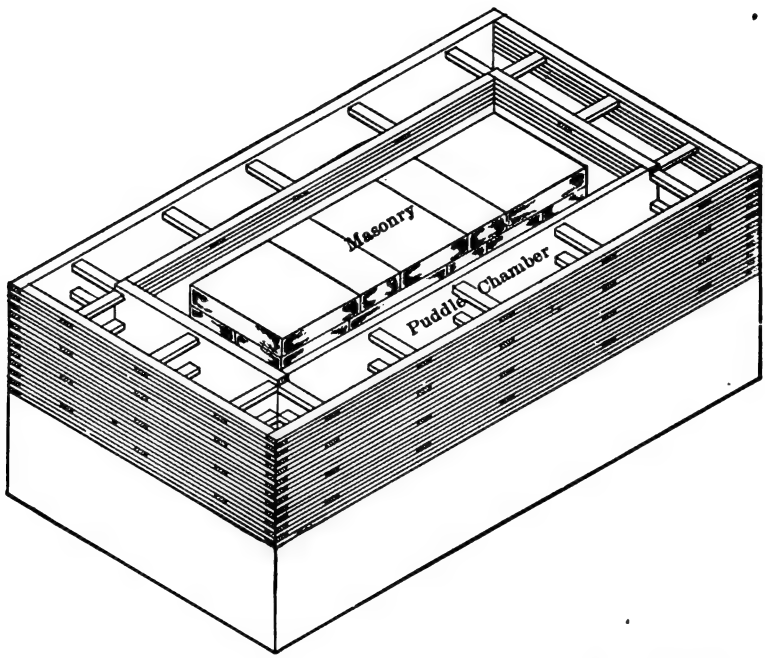


FIG. 13.—CRIB COFFER-DAM, CHICAGO, BURLINGTON AND QUINCY RAILROAD.

has been experienced except what can readily be kept under control with ordinary-sized centrifugal pumps. The cost of construction is generally a minimum, as there are usually plenty of old timbers available for use from the railroad yards.

Cribs constructed in a similar manner but with only one wall of timber have been used successfully on the Canadian Pacific Railway by P. Alex. Peterson, chief engineer.

The bracing is very efficiently attached by dovetailing it into the sides, while the form of the crib enables it to withstand the force of the current and the ice. The projections on the inside are to

prevent the water from forcing its way up between the sides and the concrete filling when the dam is pumped out. These projections answered their purpose very effectually, and when the dam was

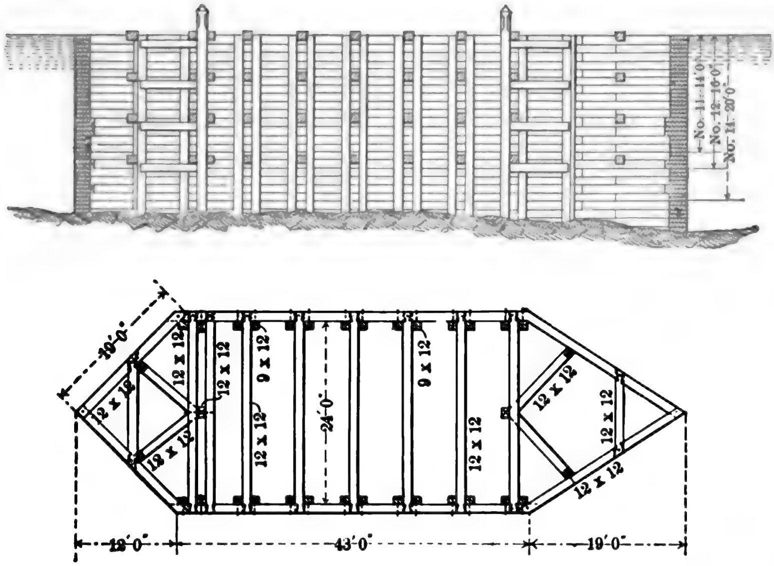


FIG. 14.—ST. LAWRENCE RIVER BRIDGE, CRIB AND COFFER-DAM, CANADIAN PACIFIC RAILWAY.

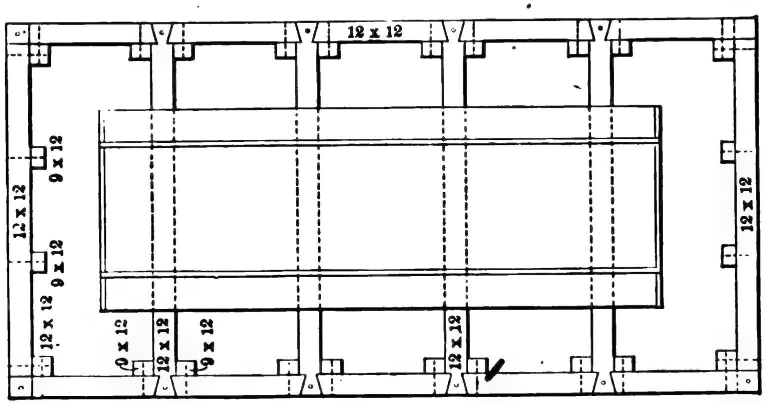


FIG. 15.—ARNPRIOR BRIDGE, CRIB AND COFFER-DAM, CANADIAN PACIFIC RAILWAY.

pumped out it remained dry enough to lay the masonry without any additional pumping.

Illustrations are given of a crib of this character which was used on the St. Lawernce River (Fig. 14), similar ones being used for the

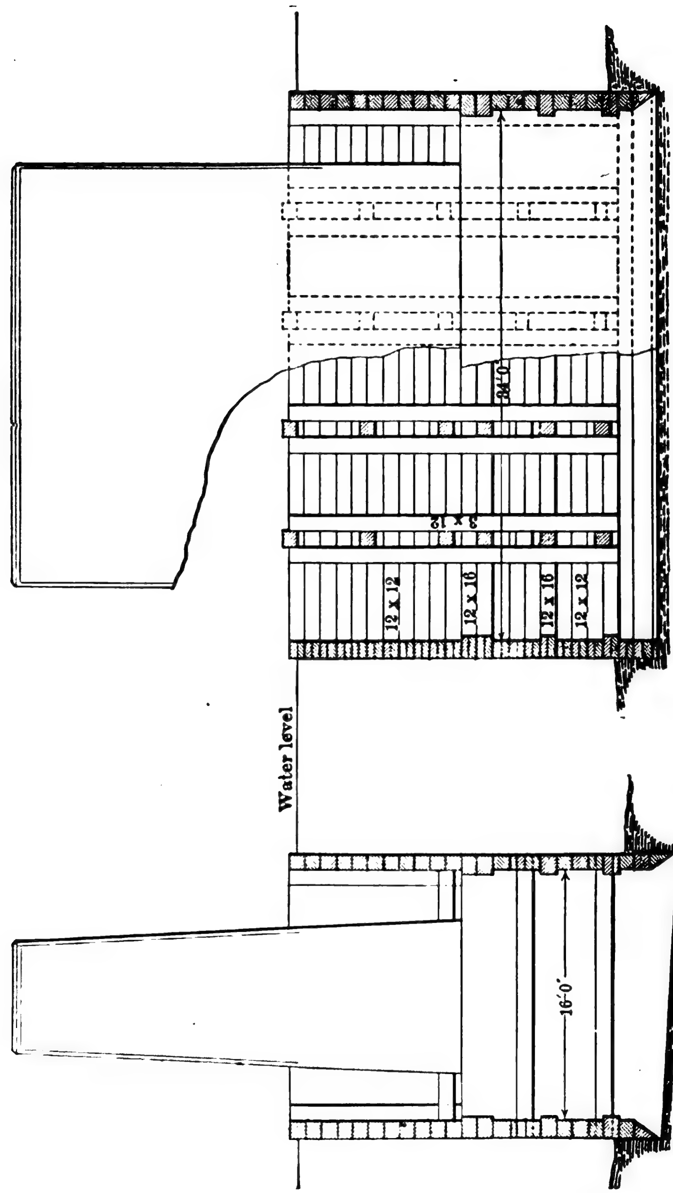


FIG. 16.—ARNPRIOR BRIDGE, CRIB AND COFFER-DAM.

other piers of the same bridge, and of the crib used for the Arnprior bridge. (Figs. 15, 16.) This shows the concrete which was deposited on which to found the masonry, and which formed a water-tight bottom so that the crib could be pumped out for the laying of the stone.

The practice on the Atchison, Topeka & Santa Fé Railroad has been in some respects similar to what has been given. C. D. Purdon, assistant chief engineer, states that cribs built of old timbers are used when such material as stringers $7'' \times 16''$ is plentiful, each course being stepped in about $\frac{1}{2}$ inch to give a batter. For use in sand when rocks and drift are likely to be encountered a crib is made by constructing a frame of old bridge timbers and sheathing it with plank. (Fig. 17.) This is sunk by digging out the sand, which is shoveled first into box *A*, then to boxes *B*, then to *C*, and then outside. The suction-pipe is shown in dotted lines, the pumping being accomplished with a centrifugal pump. This plan works very successfully on the streams in Colorado and New Mexico where the water is mostly in the sand and but little shows as surface-water.

The Arkansas River bridge of the St. Louis & San Francisco Railroad at Tulsa was built over a bottom of gravel and riprap above rock, which was quite level and about 7 feet below low water. Cribs were constructed for coffer-dams similar to the one just described and set on the bed of the stream. Clay from the bank was dumped outside and as the crib was dug out and sunk, the clay followed down and kept out the water.

When the bottom is of clay or of sand without obstructions, sheet-piles, either tongue and groove or the Wakefield, are driven around a crib.

Geo. H. Pegram, chief engineer of the Union Pacific system, has made the construction of coffer-dams conform to available material and local conditions. At the crossing of the Republican River in Kansas, where the bottom was sandy, a single thickness of 4-inch V-shaped tongue-and-groove sheet-piling, with the usual guide-piles and wales, served to form a water-tight structure.

Where a gravel bottom overlaid a hard soapstone, as on some work in Idaho, with 7 feet of water to contend with, the coffer-dam was made of Wakefield piling, formed of $1\frac{1}{2}$-inch sized plank. The joints were tightened with cement; and sand, gravel, and straw placed outside to prevent leaking. Wakefield piling has also been used for clean rock bottom, placed in two rows about the depth of the water apart. Intermediate cribs filled with rock were used to sink them. The ends of the piling were sharpened and driven on the rock until broomed up and rendered nearly water-tight, when

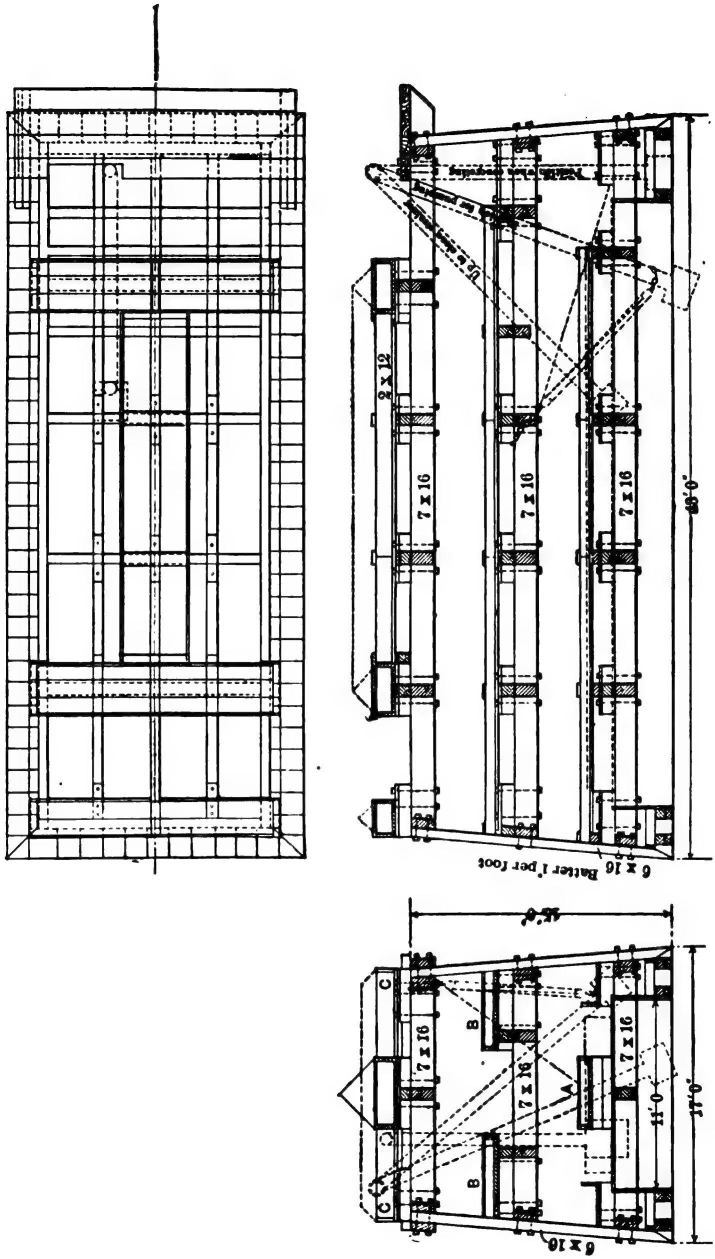


FIG. 17.—CRIB COFFER-DAM, ATCHISON, TOPEKA AND SANTA FE RAILWAY.

gravel mixed with straw was placed around outside to close any remaining leaks.

In cases where ordinary piling has been driven and a grillage laid upon them to receive the masonry, a coffer-dam is constructed as shown (Fig. 18) in which to lay the masonry. The construction of this is fully shown in the different views given.

Another form of coffer-dam for the same purpose was constructed by Octave Chanute in laying the masonry of the pivot pier for the Fort Madison bridge over the Mississippi River, on the line of the Atchison, Topeka & Santa Fé Railroad. (Fig. 19.) This is described in the *Engineering News* of June 2, 1888, by W. W. Curtis, resident engineer: "The grillage (for the pivot pier) is 4 feet 3 inches thick, the upper 15 inches being dressed to an accurate circle of the desired diameter. The coffer-dam was fitted against these two courses and was formed of $3'' \times 8''$ pine-plank staves, dressed on the sides to a slight bevel, around which were placed seven wrought-iron hoops $4'' \times \frac{3}{16}''$, $5'' \times \frac{3}{16}''$, and $6'' \times \frac{3}{16}''$, similar to those used for water-tanks, and screwed up tight. Inside of these, circular braces of plank were fitted. As a water pressure of 19 feet was to be resisted, additional security against leakage was obtained by placing a string of candle-wicking vertically between each stave. When the caisson was submerged to about full depth it became necessary for the steamboat to assist it into final position. A $12'' \times 12''$ post was bedded in the concrete in the center of the pier, with four braces running to the circular bracing of the sides. This makes a very cheap coffer-dam and was found to work very well."

An attempt to use a form similar to this was made in constructing the Walnut street bridge at Philadelphia. This is described by Geo. S. Webster, chief engineer Bureau of Surveys, in the *Engineering News* of March 15, 1894: "In founding the river piers, the Robinson coffer-dam was first tried, but was abandoned after three of them had failed by collapsing. This dam may be briefly described as follows: A circular platform about 80 feet in diameter supported upon piles at an elevation of about 4 feet above high water was first constructed. Square piles of $12'' \times 12''$ yellow pine were then prepared by spiking a $3'' \times 4''$ timber flat, along the middle of one side, and two others along the edges of the opposite side, forming a tongue and groove on each pile. The tops were squared off and the bottom ends pointed to a wedge shape. These piles were then driven close together against the edge of the circular platform and down to rock. Mr. Robinson's idea was that the mud overlying the rock would hold the piles in position at the bottom,

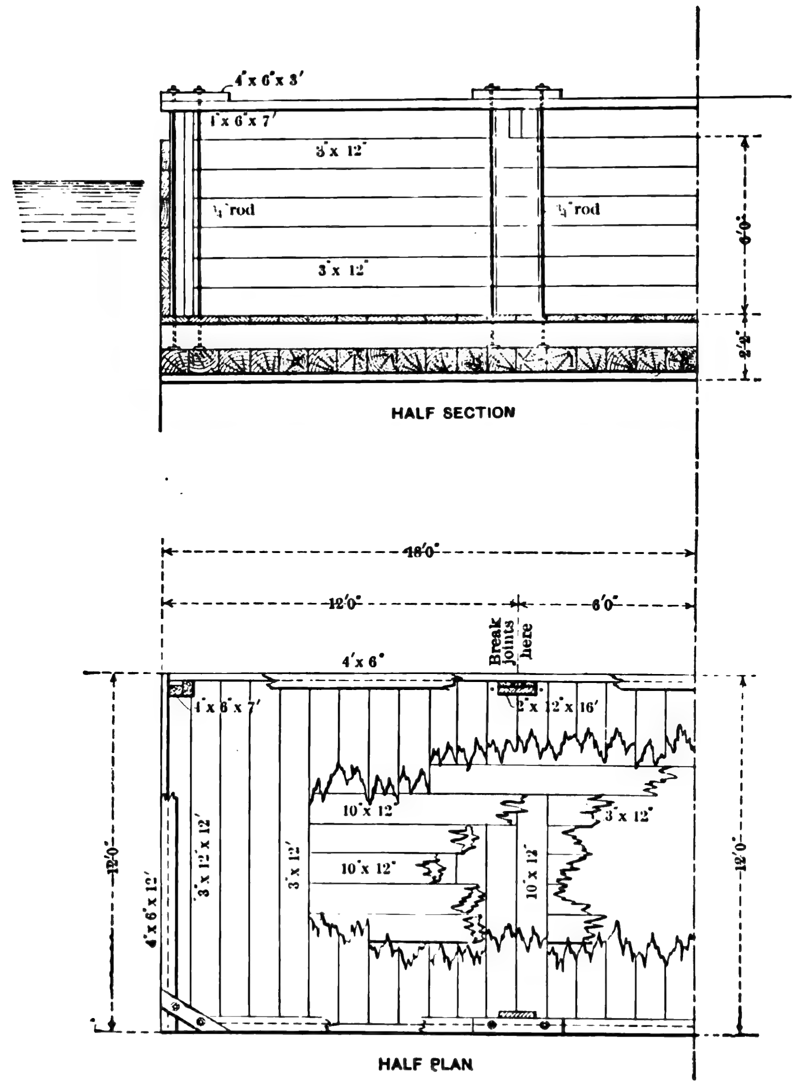


FIG. 18.—COFFER-DAM ON GRILLAGE, PAYETTE AND WEISER RIVER BRIDGES, UNION PACIFIC SYSTEM.

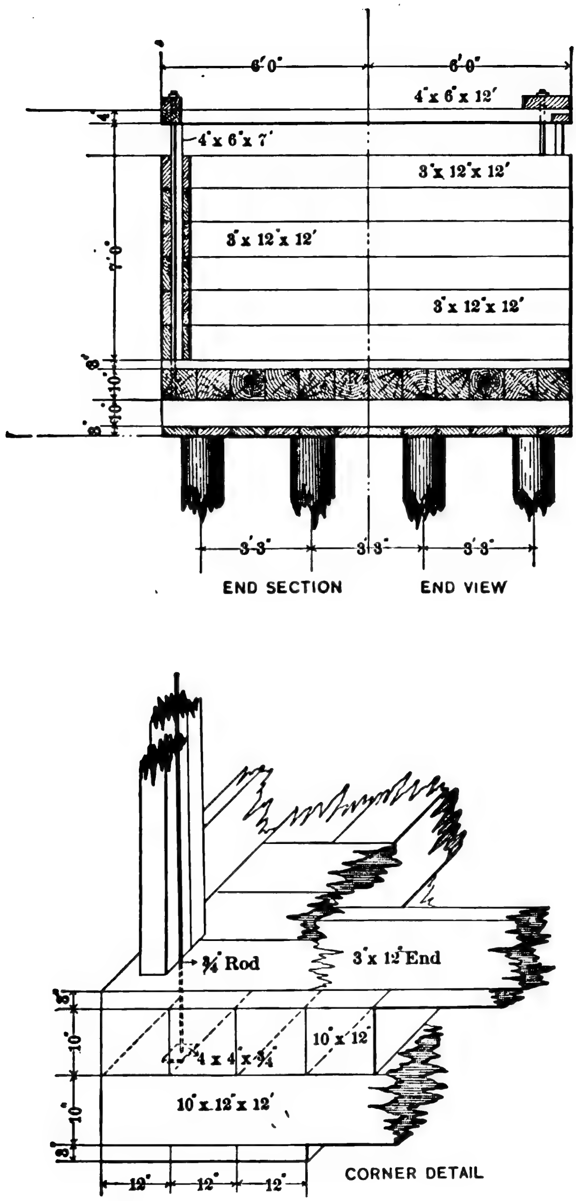


FIG. 18.—CONTINUED.

and if the top ends were held by an outside hoop, the dam would be secure without internal bracing to resist collapsing-pressure. In the first trial the hoop was made of boiler-iron some 4 feet or more in width. In the second dam it was formed of a heavy steel railway rail, and in the third dam the hoop was the same as in the second, but it also had a number of radial rods in addition. The first dam was pumped out and held for nearly an hour before collapsing, but the others collapsed before being entirely pumped out. After the third failure this form of dam was abandoned."

It would seem likely from a comparison of the two cases, one being entirely successful and the other a failure, that had the Wal-

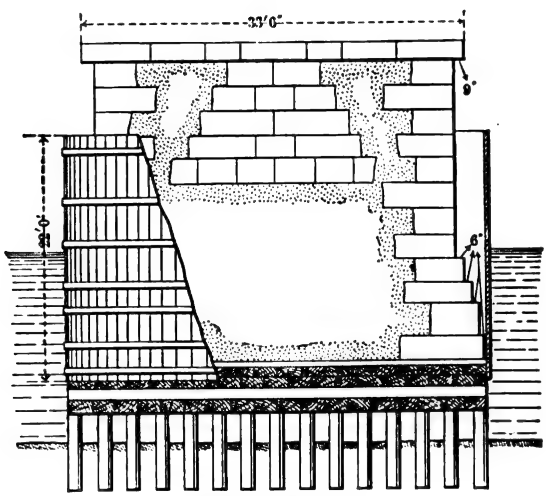


FIG. 19.—COFFER-DAM ON GRILLAGE, FORT MADISON BRIDGE, ATCHISON, TOPEKA AND SANTA FE RAILWAY.

nut street dam been supplied with additional bands lower down and provided with some means of tightening, with several internal bracing ribs of timber, it would have proven a success. These bands and ribs could likely have been placed by a diver.

The uncertainty which always exists regarding any construction under water makes it imperative that every precaution should be taken to guard against troubles that might arise, by making the construction of no doubtful form and in no doubtful manner from its first inception.

The nature of the bottom will always indicate the method of construction which should be adopted in a given case, but it would

be rarely that the preliminary dredging could be dispensed with. It is true that there are cases where there is a deposit overlaying a seamy rock, and the water will find its way along the seams, bubbling up in springs inside. Recourse must be had to cutting off the flow, by puddling on the outside, sometimes extending the operations a distance of a hundred feet or more away, until enough of the flow has been stopped so that the water can be kept down by a reasonable amount of pumping.

The next precaution after dredging is the building of some form of coffer-dam which shall effectually exclude any flow through the

Fig. 20.—A Crib Coffer-dam after a Flood.

sides of the dam. This we have seen to be accomplished in many cases by means of a bank of clay, or a row of sheet-piling, and in some cases by a single-walled crib. But in the last two methods a supplementary bank of clay or clayey gravel on the outside is necessary to prevent leakage.

This bank may be protected from wash by covering it with clay, sand, or gravel in gunny-sacks, or by riprapping up to about low water, as was done on the Kanawha dams.

Double-walled cribs and coffer-dams, constructed with two rows of water-tight sheet-piling, require to be puddled with a carefully selected material. While clay can be used with a good degree of success, it will be found better to use a clayey gravel or to mix the

clay and gravel, as was done at the Buda Pesth bridge. When a small leak starts through a pure clay puddle, it washes out the clay in considerable quantities and a dangerous leak is soon developed. With the admixture of gravel, however, a leak is stopped almost as quickly as started by the heavier gravel falling into and closing the void.

It will generally be found advantageous to use a bank of clay outside of a double-walled dam, unless it might be a case where sheet-piling has been driven to rock, and even then a certain amount of material in sacks should be used to prevent wash or the cutting out of the earth around the sheeting.

Whatever excavation is taken out of the interior of the coffer-dam after it has been pumped, should be dumped at the up-stream end and corners, or to fill any holes or pockets there may be around the sides or ends.

Cutwaters should be added to all coffer-dams which are built in rivers having a swift current or a heavy flow of ice, as was the case at Buda Pesth and on the Canadian Pacific examples. They must also be used in rivers where the run of drift with each rise is of large amount. For the purpose of preventing wash around a dam, a cutwater of plank supported by a frame of timber may be constructed separate from the main structure, or a V-shaped row of sheet-piling driven up-stream. On rock, a timber crib of triangular shape, built of round logs, may be sunk up-stream and filled with broken stone. Such a crib can be utilized in anchoring the main crib of a coffer-dam, as was done at St. Louis, and which will be described in future pages.

The use of a log crib by the author, somewhat similar to those used on the Great Kanawha River, was employed in placing a reinforced concrete pipe 112 inches in exterior diameter under the bed of Green River for the Tacoma, Washington, water system.

The location (Fig. 21) was at a narrow point in the river where there was considerable fall in a short distance, and the first cribs enclosed about one-third of the width of the river.

The logs were halved into each other and drift-bolted together with $\frac{3}{4}$-inch drift-bolts; the filling was composed of the muck from a tunnel on one side of the river immediately adjoining, comprising broken rock, rock dust, gravel and some red clay. The coffer-dam having been set directly on the bed of the river without any previous excavation, there was some seepage underneath the dam at the junction with the river bed. To take care of this an inside sack dam was built, to carry this water around to the low side of

the coffer-dam, where the fall of the river in that distance made it possible to discharge it into the stream.

The coffer-dam had a height of from 4 feet near the shore to 6 or 8 feet out in the stream, and for probably two-thirds of the depth excavated, a 6-inch centrifugal pump kept the water out, but as the depth increased up to 15 to 18 feet the water came in through the pervious gravel bed of the river to such an extent that an additional 6-inch pump, operated by a 16 horse-power gasoline engine, was required to keep it dry enough to work in.

FIG. 21.—GREEN RIVER LOG CRIB COFFER-DAM.

On the river side some sand bags and some Wakefield sheet-piling were used to prevent caving in of the excavation.

The placing of the pipe (Fig. 22) was accomplished by setting the circular inside forms on concrete blocks, which were buried in the concrete of the pipe when it was poured. Then the reinforcing was placed and wired together and the outside forms set ready for the concrete, which was deposited from the mixer directly on the work by wheelbarrows, the water being excluded except a small portion at the bottom and which was forced out ahead of the concrete as it was poured.

The balance of the river on the other side was included in the

other portion of the coffer-dam, after the pipe had been back-filled and riprapped, this allowing the river to be turned through. The cost of this work will be given in the chapters on cost at the end of this volume.

More fitting language cannot be found for closing words than those used in Wellington's monumental work on railway location: "The uncertainty as to the exact requirements to be fulfilled by the works when completed is a disadvantage, indeed, which cannot be escaped;

FIG. 22.—PLACING REINFORCED PIPE, GREEN RIVER COFFER-DAM.

but the more difficult it is to reach absolute correctness, the greater need we have of some guide which shall reduce the unavoidable guess-work to its lowest terms, and so save us from the manifold hazards which result from not only guessing at facts, but at the effect of those facts. Whatever care we use we can never attempt with success to fix the exact point where economy ends and extravagance begins; but what we can do is to establish certain narrow limits in either direction, somewhere within which lies the truth, and anywhere outside of which lies a certainty of error."

CHAPTER III

CONSTRUCTION AND PRACTICE—CRIBS AND CANVAS

When for some reason the necessary care was not exercised in the construction of a coffer-dam and in puddling it, or where there were discovered conditions not known before the construction began, which rendered the work unsatisfactory or leaky, it will usually be found that the mode of repair which seems most expensive will in the end prove the cheapest and most expeditious. If the puddle proves leaky, and it be decided that the material was of too porous a nature, the best remedy is to dig out and replace it with better. Should it be found that the porous bottom had not been removed to a sufficient depth, it may be found necessary to dig out the puddle-chambers and puddle deeper, or the leaks might be stopped by banking up outside of the dam with clay or clayey gravel, or perhaps sand in sacks would do some good.

Gravel will allow the percolation of water even where the head is small, and when a pressure of from 4 feet upwards is brought upon it, the leakage becomes considerable and difficult to control, so that pure gravel is of little service in stopping leaks.

Hay, straw, oats, crushed cane-stalks, rotten stable manure, and similar materials, mixed with the banking material, are very efficacious in producing tightness, and when applied to local leaks will assist in closing them.

Where sheet-piling has been used to exclude the water and leaks still occur, they can often be closed by driving more sheeting to lap the cracks, which may have been widened out lower down as the sheet-piles were first driven. This, we have seen, produced satisfactory results at Buda Pesth, where leaks were also closed by driving square timbers into the puddle to compact it.

Clay can also be forced down through pipes directly to where the leakage occurs. The use of this at the Government Lock at Sault Ste. Marie is described in the *Engineering News* of September 26, 1896: "The only difficulty encountered in the work of excavation was due to a leak in the coffer-dam, which flooded the lock-pit and

delayed the work considerably. The cause of this leak was found to be a crevice in the rock passing underneath the coffer-dam, and despite all efforts to close it, the flow of water rapidly enlarged the break until about 50 feet of the clay in the coffer-dam had been washed away. The large break was closed by driving additional sheet-piling and filling in with brush, hay, and clay in sacks. This, however, failed to entirely stop the leak through the crevice, and it was determined to fill the cavity with clay. For this purpose a 3-inch pipe was driven down through the coffer-dam until its lower end penetrated the crevice. In this pipe small cylinders of clay about one foot long were placed and forced down into the cavity by means of a plunger working in the pipe. The apparatus is shown in the illustration (Fig. 23). As will be seen, the plunger, or rammer, is an iron rod, to the top of which is fastened a block of wood sliding between the guides of an ordinary pile-driver. The hammer of the pile-driver is the weight which pushes down the rammer. This apparatus was designed by E. S. Wheeler, engineer in charge of the work, and was used not only to fill the crevice, but all along the coffer-dam for the purpose of compacting the clay filling. The apparatus proved most successful for the purpose for which it was intended."

The use of rods for bracing in double-walled coffer-dams is very often the cause of considerable leakage, the water following along them through the puddle. This may be stopped by wrapping a band of hay or straw around the rod next to the timbers, or by a wrapping of coarse cloth, or by a wood washer having a hole slightly smaller than the rod, which is forced through.

The walls of the dam must always be made tight, and this we have seen to be effected by careful framing of sides and bracing, and it will be seen in a later example how round struts between the two walls allowed the puddle to flow around them and close up much better than if the braces were square timbers.

The use of candle-wicking between the staves proved successful at Fort Madison, and calking is very often resorted to at the first, and also to close up local leaks. The use of this and the use of a stiff grease between the layers of a crib will be referred to in another part of this article.

The use of tarpaulins to make a water-tight piece of work is described in the Trans. Am. Soc. C. E., Vol. 31, by Montgomery Meigs, engineer in charge of the government work at Keokuk, Iowa. "The upper one of three locks was twice repaired by separating it from the river by an ordinary plank and mud coffer-dam. But as this

work had to be done after the close of navigation, it was found to be very unsatisfactory on account of the freezing of the puddle, and on one occasion the partly puddled dam froze and upset. After this experience it was determined to use some other method than

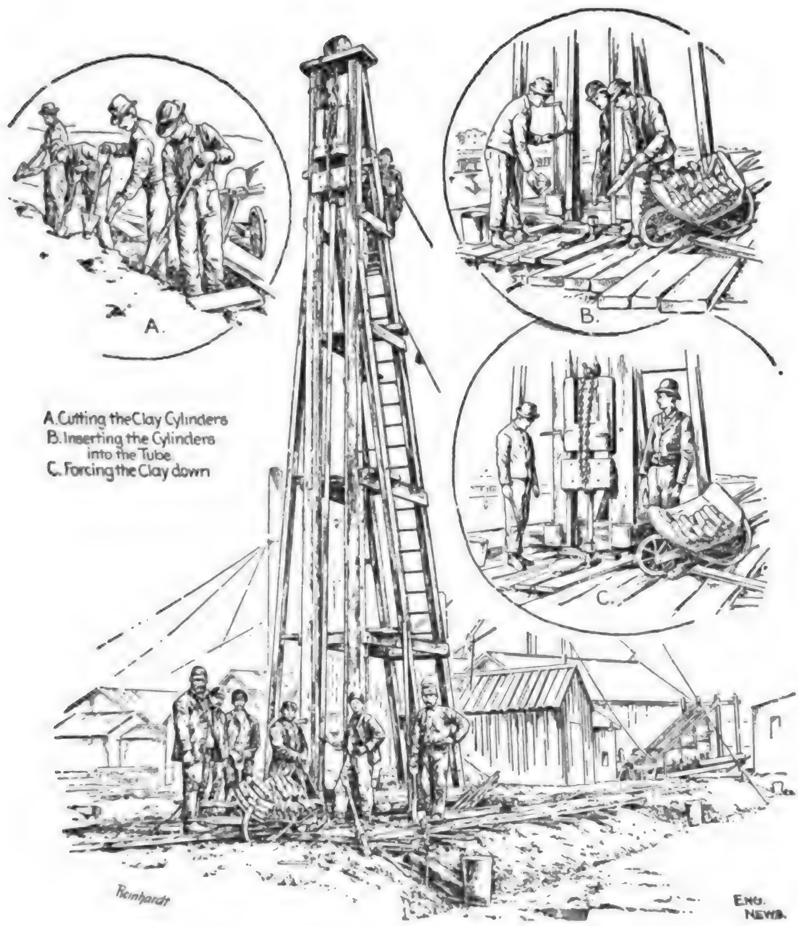
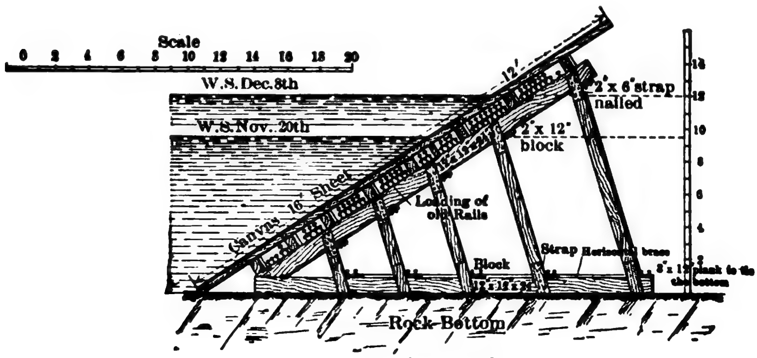


FIG. 23.—APPARATUS USED TO FORCE CLAY INTO CREVICE OF FOUNDATION ROCK AND CLOSE LEAK IN COFFER-DAM.

puddle to produce tightness. There was available for drainage a 50-H.P. suction dredge, with 14-inch suction, and a rotary Van Wie pump, and plenty of 12-inch discharge-pipe mounted on pontoons. It was proposed to drain the lock with this dredge, allowing the boat to settle in the mud at the bottom of the lock as the water

left it, and to complete the work with a 3-inch discharge pulsometer. The lock being 350 feet long and 80 feet wide, a flat place on the bottom was selected, the dredge placed over it and the necessary length of discharge-pipe placed in position on its pontoons. The point selected for a bulk-head (Figs. 24 and 25) was just outside the lock gates, about 40 feet below the lower miter-sill, where there



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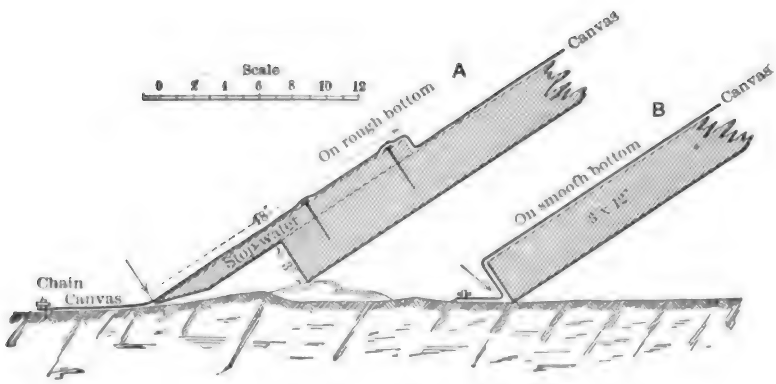


FIG. 24.—DETAILS OF CANVAS AND PLANK BULKHEAD.

was a smooth rock bottom, the ends of the dam abutting against the flaring ashlar wing-walls of the lock approach.

"The bulkhead was constructed with thirteen bents 8 feet apart, of the size timber shown, with light diagonal bracing. After being built $2\frac{1}{2}$ miles from the lock it was towed to position and sunk by weighting it with old railroad-rails, enough being used to overcome the buoyancy after the sheathing was added. A diver was employed to see that the bottom was clear of obstructions and to guide the

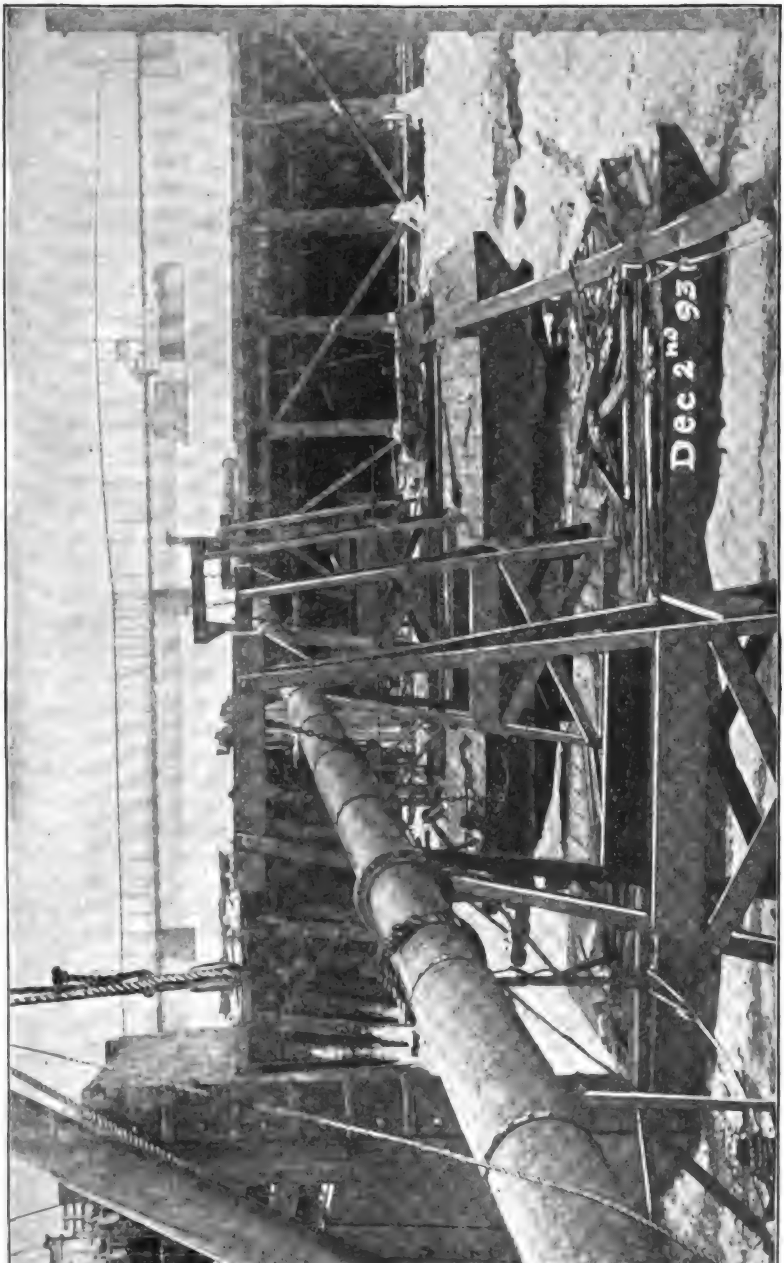


FIG. 25.—INSIDE VIEW OF BULKHEAD. LOCK PUMPED DRY.

bulkhead to a solid bearing. The sheathing was also guided to place by his assistance.

"The canvas sheet, which was designed to give tightness to the apron, was of two breadths of 10 feet and one breadth of 6 feet wide, sewed together edge to edge for convenience, and about 4 feet longer than the extreme length of the apron. Some old $\frac{1}{2}$-inch and $\frac{5}{8}$-inch chain was sewed to one edge continuously to act as a sinker and insure the lower edge of the canvas sheet hugging the bottom tightly. A few stones laid on it would have answered the same purpose, but not so well. The canvas was 12-ounce duck.

"The sheet was spread under water by the diver. It lapped on the bottom about 12 inches, covered the face of the apron and extended some inches up the face of the wing-walls at the end of the dam. Cleats were nailed on the angle between the apron and the wing-walls. These were of 1×4-inch strips, nailed with 2-inch wire nails about 12 inches apart. The upper edge of the canvas was also lightly cleated to the planking in a similar manner. No other nails were driven in the canvas, which was designed to be cut up into tarpaulins eventually. Where the plank touched bottom no beveling was used, but one ragged hole was stopped with the beveled 'stop waters' which were made use of. The dam was pumped out in about 6 hours and the leakage was so small that a 3-inch discharge pulsometer kept out the water, and was then run only at intervals. Small leaks were stopped by dumping rotten stable manure in their vicinity."

It is interesting to note that the bulkhead stood a pressure of 12 feet of water. Experiments made to determine what pressure 12-ounce duck would stand, show that the clean canvas begins to leak at 2 pounds pressure, and at 5 pounds pressure the leakage becomes a marked amount. With mud on the canvas the leakage becomes noticeable at from 5 to 7 pounds, and of a considerable amount at 50 pounds pressure, these pressures being on a circle $4\frac{1}{2}$ inches in diameter. The canvas did not rupture at 800 pounds.

The suggestion is made to use an inverted funnel of canvas to stop the leakage of springs on rock bottom. (Fig. 26.) The canvas to be spread out over the bottom and weighted down with concrete, and the top wired to a pipe into which the water may rise until the pressure-head is overcome or the pipe can be plugged. Arrangements of this nature, but without the canvas funnel, have been frequently used. An iron pipe set on end is fitted over the leak, and after concreting around to make it water-tight, the water rises inside until the pressure is balanced. A water-tight wooden box may also be used for the same purpose.

The founding of a new inlet tower in the Mississippi at the St. Louis water-works was accomplished by using a coffer-dam, and it was the intention to form a junction with the bottom by using a canvas curtain. When the coffer-dam was floated into position and the divers were sent down to spread the canvas and weight it down with stones, it was found to be damaged so as to be useless. This was supposed to be due to the action of the swift current, but was most probably due to some accident such as fouling on a snag or against a barge.

The anchoring of the crib for this dam is related in the *Engineering News* of July 4, 1891. The dam was to be located near the head of a stone dike about 20 feet in height and on solid rock bottom which was uneven and worn into grooves by the action of the current,

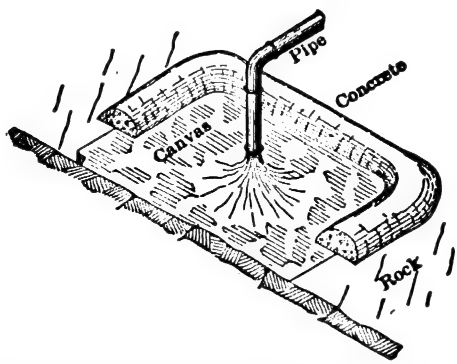


FIG. 26.—CANVAS FUNNEL FOR CLOSING LEAKS.

which had a velocity of between 6 and 8 miles per hour. The bottom was leveled off by blasting, to receive the crib, which was to be sunk in from 15 to 18 feet of water.

The three triangular cribs shown (Fig. 27) were sunk ana filled with stone and were used to hold the dam in place while building and while being sunk. Steel cables $1\frac{1}{2}$ inches in diameter were used as anchors.

The large crib also served as a protection from the current and drift.

The size of the crib was 38×74 feet outside and the height 22 feet. The 12×12-inch yellow pine timbers were drift-bolted together with from 1 to 2 feet spacing of bolts, and all the joints between the timbers were calked. The bracing consisted of 12-inch square timbers, of which there were three rows, the braces in each row being 4 feet

apart vertically. These were cut out as the masonry was built up and bracing against the stone work substituted.

There were four sets of diagonal bracing as shown. The space between the walls, which was 3 feet, was partly filled with concrete in sacks, and puddle placed on top. Sacks of clay were banked up around the outside, and then the dam was pumped dry with a 10-inch pump. Inside was found 8 feet of mud and 60 sacks of concrete which had been washed there by the swift current.

The amount of timber used was 125,000 feet, B.M., and about 12,000 feet of $\frac{7}{8}$-inch round iron for drift-bolts. The puddle-chamber required 1000 sacks of concrete and 100 barge loads of clay,

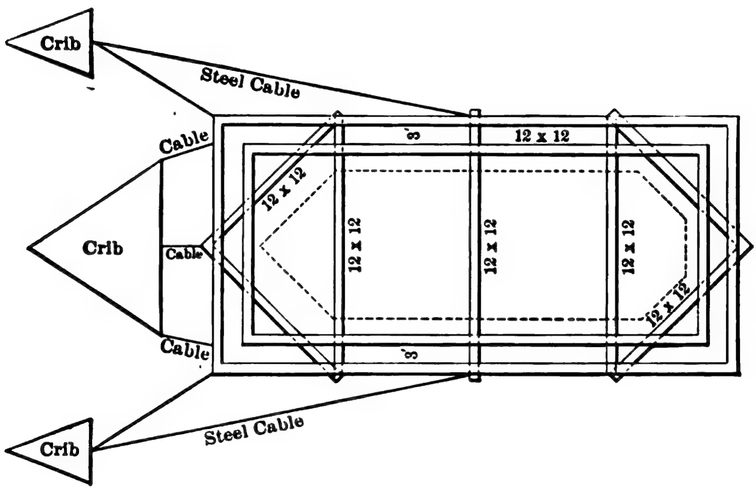


Fig. 27.—Cribs for Anchoring St. Louis Coffer-dam.

while 10,000 sacks were used for banking up clay on the outside. This work was constructed under the direction of C. V. Mersereau, Division Engineer, under S. B. Russell, Principal Assistant Engineer.

The Queen's Bridge at Melbourne, Australia, is a plate-girder structure with four piers of 8 cylinders each. The bottom was a reef of bluestone which had been shattered by blasting and which was silted over with about 3 feet of very soft silt.

The use of ordinary puddle coffer-dams was thought to be too expensive, as the bridge was 100 feet in width, and it was proposed to use a single wall of timber protected by tarpaulins. The account of this work is taken from the *Engineering News* of April 4, 1895, which is an abstract of a paper by W. R. Renwick, engineer in charge.

To insure as light a construction as possible experiments were

made on the strength of Oregon pine, and it was found that tests of water-soaked timber showed a loss of strength of as much as 33 per cent., when compared with tests of seasoned timber. The break, too, of the water-soaked pieces was very short. This strength being the one adopted, a very low factor of safety was used. A separate dam was constructed around each tube, but with one side to open as a door to allow its removal and use for another place. The frame was made from 12×12 Oregon pine, with the sticks placed closer together near the bottom to resist the greater water pressure, and 12×12 pieces were run up the corners, the frames being notched in. These also served as spacers for the side timbers and as door frames. The sheeting on the outside was of 4×12 rough timber, and outside of this at the top and bottom were wale-pieces, 6×12, bolted through the frames with 1-inch bolts to hold the sheeting in place.

The tarpaulin was passed completely around the dam, being tacked to the waling-pieces, and so arranged as to allow the door to open.

When the dam had been placed around a tube the sheeting was driven down to rock, through puddle which had been dumped on the bottom, and the pumping was readily done with pulsometer pumps. The only serious leaking was where the 1-inch bolts passed through the joints between the sheeting, but these were plugged with soft wood plugs, and in other work the bolts were flattened to three-eighths of an inch where they passed between the plank. The dams were removed by first drawing the sheeting up to its original position, when the door was opened and the crib taken to another tube. The depth of water was about 15 feet, but while this was successful in this instance, the method should not be copied unless the conditions are favorable, nor unless the cribs are made practically water-tight in themselves.

This was the case in the above work, as one of the tarpaulins was accidentally torn off and the dam still excluded the water, so that the tarpaulin was only a wise precaution. Why the cylinders were not made water-tight and used as their own coffer-dam is not stated, but this possibly could have been done.

Sheets of tarpaulin in closing accidental leaks could doubtless be employed frequently, but as the sole dependence for producing tightness it should be used with extreme care, in a gentle current and well protected from damage.

The pivot pier of the Harlem Ship Canal bridge was founded in a polygonal coffer-dam, from the plans of William H. Burr, consulting

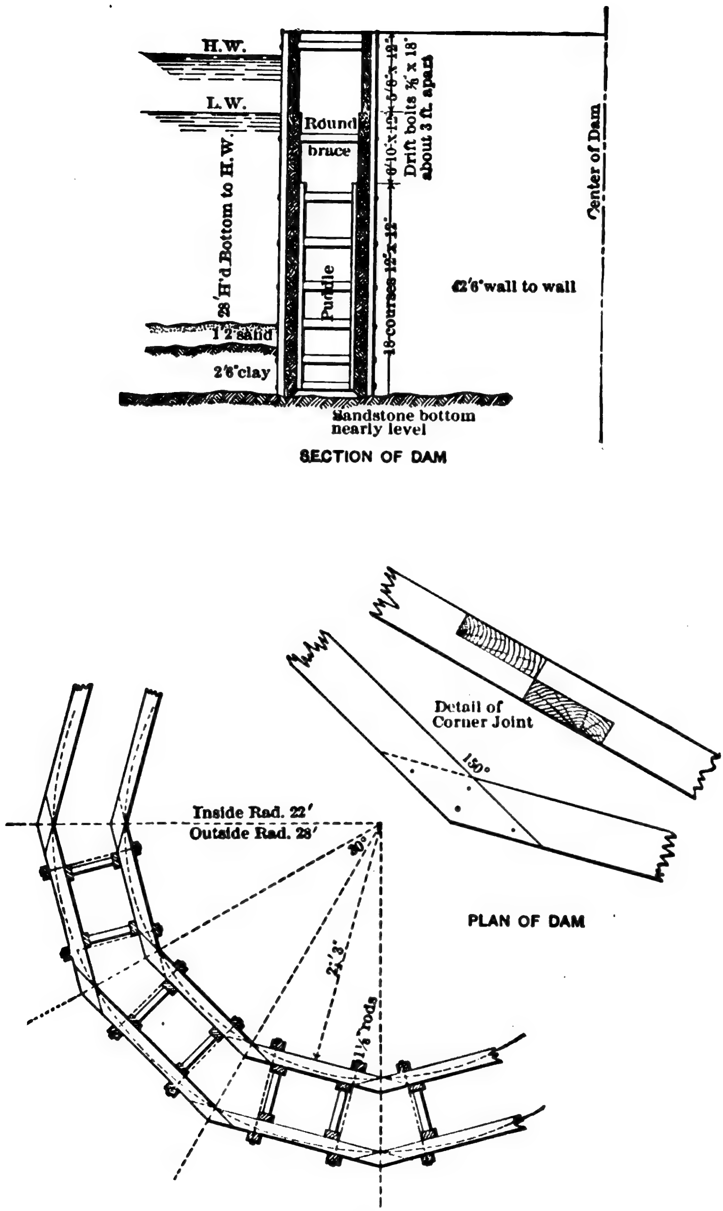


FIG. 28.—DETAILS OF COFFER-DAM USED ON ARTHUR KILL DRAWBRIDGE.

engineer. The work is described in the *Engineering Record* of July 24, 1897: "The rock bottom secured by the canal excavation being an acceptable surface for the masonry of the pivot pier, it was constructed in a polygonal double-walled coffer-dam with thirteen sides 25 feet high and 60 feet in extreme diameter. The great dimensions of the coffer-dam would have made it difficult to build and launch it on shore. Consequently it was built partly on a detachable raft. As shown in the illustration (Fig. 29) the inside wall was built up of timbers lapped and halved at the angles; the outer wall timbers were carefully butt-jointed and secured by cross-struts and 1-inch bolts to the inside walls. The rough-sawed horizontal surfaces of the inner wall were bedded in stiff grease and the joints calked, which notably resisted the penetration of the water. Each course of timber was secured to the one below it by ¾-inch drift-bolts spaced about 4 feet apart. When the bottom was thoroughly cleaned the concrete was dumped in place by a special steel bucket. Concreting was carried on night and day and was completed before puddling was begun. Considerable difficulty was occasioned by the irregularities of the bottom which the coffer-dam could not be made to fit closely. Divers were sent down and filled in bags of sand, as at *S*, and riprap *R* was piled up outside to protect it. Then the space between the walls was filled with puddle."

Another polygonal dam was constructed for the draw pier of the Arthur Kill bridge, by Alfred P. Boller, consulting engineer. The following account is taken from Vol. 27 of the Transactions Am. Soc. C. E.: "It was necessary to use as little space as possible for the dam, and to construct it without interior bracing, so that a double-walled twelve-sided polygon (Fig. 28) with walls 4 feet apart in the clear was used. The rock bottom was over-laid with 2 feet of clay and the clay with 18 inches of sand and mud, the depth of water over the rock being 28 feet at high tide. The square hemlock timbers used in the walls were halved together and the walls braced together by bolts and round timbers for struts, the round timbers allowing the puddle to run around them and pack well as thrown in. Clamp timbers 4×6, in two lengths, were held in place by the bolts and the struts were braced against 6-inch plank. The dam was built to one-third its height on shore, then towed to position and built up until grounded. Between the timbers and the joints candle-wicking was placed, and the courses drift-bolted together every 3 feet and spiked at the joints. The rock was dredged bare before placing the crib, which was filled with a hard, gravelly clay between the walls after being sunk in place. A rich Portland concrete was dumped inside,

from triangular buckets, to seal the bottom, and then the dam was pumped out with a 6-inch pump and kept dry by pumping at intervals. In one place the concrete was not thick enough and a spring came up through a fissure in the rock. This was boxed in and led to the sump. The material used was 140,000 feet of timber, 15,000 pounds of iron, and 600 yards of puddle."

A piece of work similar to the Canadian Pacific example was an octagonal single-walled dam used in the construction of the Coteau

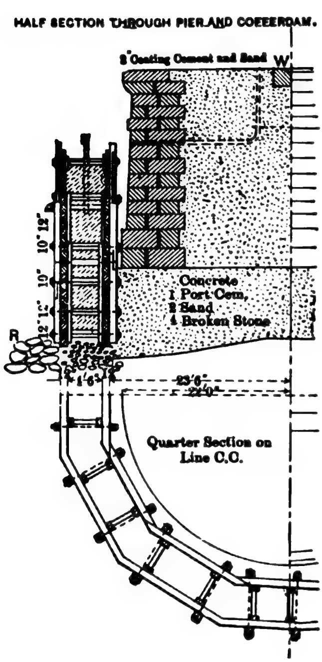


Fig. 29.—Polygonal Coffer-dam, Harlem Ship-Canal Drawbridge.

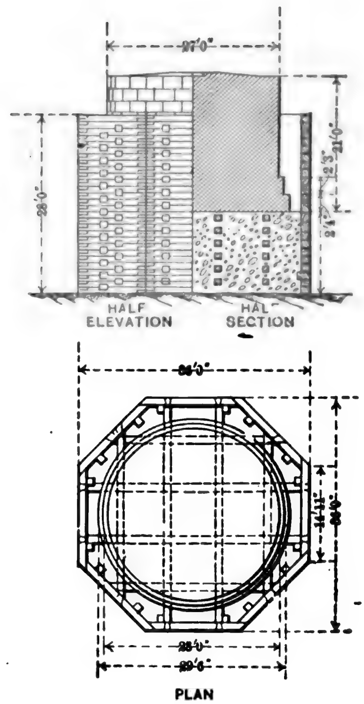


Fig. 30.—Coffer-dam for Pivot Pier of the Coteau Bridge.

bridge on the Canada Atlantic Railway. This is illustrated in the *Engineering News* of May 30, 1891 (Fig. 30). The dam was braced thoroughly with cross-timbers built into the sides, and the bottom being of rock it was partly filled with concrete to make it water-tight.

The following account of a very cheap and novel method of construction is from the *Engineering Record*, Aug. 2, 1913:

" Chelsea Bridge North carries highway and street-car traffic

over the north or main channel of the Mystic River, between the Charlestown District (Boston) and Chelsea, and forms a part of one of the most important highways out of Boston. It has pile approaches, built in 1880, and had a retractile drawspan built in 1895 and lengthened in 1900 to span a waterway 60 ft. in width.

"On account of the rapidly increasing water traffic of the Mystic River, and because the waterway was in an unsatisfactory location, the Secretary of War, on January 3, 1910, ruled that the bridge was an unreasonable obstruction to the free navigation of the river and ordered that the clear width of the draw opening be increased to 100 ft. or more.

"The new drawspan is to afford two waterways, each 125 ft. wide in the clear and 30 ft. deep below low water. It will be 363 ft. long over all, 60 ft. wide, and will weigh 1400 tons, affording a clearance above mean low water of 25 ft. The contract for building the permanent pivot pier and the wooden fender piers, for the rebuilding of the existing pile approaches, and for building and removing the temporary by-pass bridge to provide for travel during the reconstruction was awarded in February, 1912, to Mr. George T. Rendle, of Boston, and will cost approximately $200,000. The estimated total cost of the whole work, including the draw superstructure, is $425,000.

"The pivot pier has a concrete base 60 ft. in diameter and 39 ft. high, with its foundations on solid rock and the upper part about 1 ft. above mean low water. The weight of the base and the volume of concrete required for it are reduced by the construction of a concentric cylindrical chamber 30 ft. in diameter and 22 ft. high in the upper part. The top of this chamber is spanned by steel I-beams supporting part of the load from the pier shaft above, which is 51 ft. in diameter and about 13 ft. high from the top of the base to the top of the coping. It is faced with five courses and a coping of quarry faced granite, backed by a solid mass of concrete, which, like that of the base portion, is made of 1 : 2 : 4 Edison Portland cement, sand, and stone up to 2 in. in diameter.

"At the pier site an excavation 65 ft. in diameter was dredged to bedrock, which was found at depths varying from 30 to 38 ft. below low water. The material was removed chiefly by a dipper dredge and consisted of about 8 ft. of soft silt mixed with sand, from about 17 ft. to 25 ft. below low-water level, beneath which there were about 8 ft. of blue clay, and then a stratum of very hard sand, gravel and clay, with a thin stratum of shale covering the bedrock. The slopes of the excavated area were maintained at 2½ horizontal to 1 vertical.

"The basket crib, Fig. 31, or form for the pier foundation, was built of about one hundred and forty-five horizontal courses of 3×12-in. yellow-pine planks, 8 ft. long, laid flat and breaking joints. The ends were beveled to make radial joints, and each plank was secured to those below it by 1-in. oak treenails 9 in. long, two at each end of each plank, about seven thousand eight hundred treenails being required for the entire crib. In addition the planks were well spiked to the lower courses throughout their entire length with 6-in. spikes. The courses were also secured together by 4×12-in. vertical planks opposite alternate joints, which were fastened to the inner circles of the crib by lag screws. The crib, which contained about 83,000

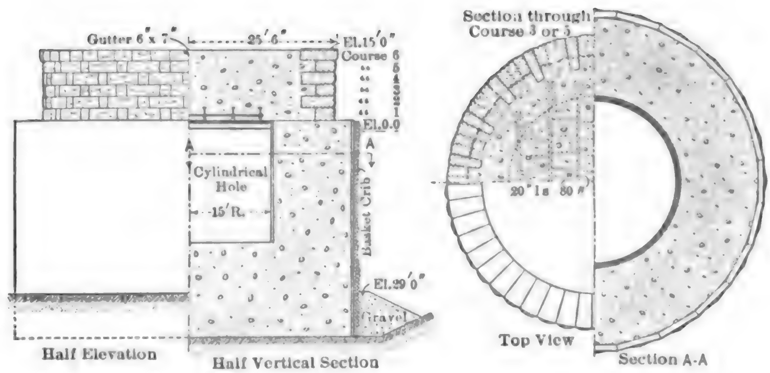


FIG. 31.— COFFER-DAM FOR PIVOT PIER, CHELSEA BRIDGE.

ft. B. M. of yellow-pine plank, was built up to a height of 3 or 4 ft. on shore between high- and low-water marks, and was then floated to deeper water and completed while still floating. After completion it was towed into position and held by guide piles, spaced about 12 ft. apart around its circumference.

"To sink the crib it was first planned to build exterior pockets which would be filled with gravel, but they were eventually dispensed with, and the crib was sunk by loading it with old iron, with stone intended for use in the pier masonry, and with heavy chains hung over the walls of the crib.

"There was no attempt to construct the crib so that on the bottom it should conform to the variations of the rock surface. Instead, the bottom of the crib was made level and it was sunk until it took bearing on only a portion of the lower edge at the highest rock level. Then, to provide continuous bearing at all parts of the circumference, and especially to complete the inclosure of the crib and confine the

concrete that was afterward deposited within it, wooden boxes of varying size, but averaging about 4 ft. square, and 4 ft. deep, were filled with lean concrete, lowered to the bottom and placed by divers under the edge of the crib to form a continuous wall. After the concrete boxes were placed the excavation outside of the crib was backfilled with gravel and dredged material until the whole crib was surrounded by filling to about 29 ft. below low water, or some 2 ft. above the bottom course of plank of the crib. This back-filling formed an effectual seal to retain the concrete which was deposited in water inside the crib without unwatering the latter.

"Broken stone and sand were delivered both by lighter and by team. The cement was conveyed by team. All the materials were stored on the old bridge close to the draw opening, where the stone and cement were measured by shoveling into scale boxes. Between the old bridge and the pivot pier was moored a concrete mixing scow, equipped with a Milwaukee mixing machine and a boom derrick. The derrick delivered the scale boxes of sand and stone from the old bridge to the charging hopper of the mixing machine. The concrete was mixed in 1-yd. batches and delivered to a submarine bucket with bottom flap doors. The bucket was handled by the derrick on the mixing scow and was so constructed that the doors were not opened until the bucket was seated on the bottom of the pier foundation, after which the concrete was automatically deposited as the bucket was lifted by the derrick.

"After the concrete foundation had been built up to within 20 ft. of low-water level a permanent form was set in the center of the pier for the cylindrical chamber above referred to, and the remainder of the concrete was deposited in a concentric ring between this form and the basket crib. The concrete was deposited in water up to about low-water level, and as the basket crib extended about 5 ft. above low water it was possible to unwater the crib at half tide or less, and the remainder of the work on the pier was carried on in the dry. This consisted in placing 20-in. steel I-beams over the hollow center, the building of forms upon these I-beams, and the covering of the whole pier foundation with a layer of concrete about 2 ft. in depth, which was leveled off in readiness for the laying of the stone masonry of the upper portion of the pier.

"The pier contains about 3673 cu. yds. of concrete and 323 yds. of granite masonry. The construction of the basket crib was commenced Aug. 8, 1912. It was sunk in position Sept. 14, 1912. Concreting was commenced Sept. 28, 1912, and the pier was completed ready to receive the superstructure Dec. 7, 1912.

"The work was designed and executed under the direction of the Public Works Department of the City of Boston, of which Mr. L. K. Rourke is commissioner; Mr. Frederic H. Fay, engineer, Bridge and Ferry Division, and Mr. S. E. Tinkham, engineer of construction."

The different forms of sheet-piling will next be taken up, together with the pile-driving machinery and the methods of driving both sheet- and guide-piles. After this will be described the use of sheet-piles for forming water-tight coffer-dams, by reference to actual constructions of that character.

CHAPTER IV

PILE-DRIVING AND SHEET-PILES*

In no department of engineering have ancient methods been more rigidly adhered to than in that of pile-driving. The form of the pile-driver derrick has remained so characteristic that a person but slightly familiar with the subject would have little difficulty in recognizing the pile-driver in the picture of Cæsar's Bridge (Fig. 3) in Chapter I. The bridge of the Emperor Trajan over the river Danube is an instance of the early use of piles. This bridge was constructed in the first century, and when the piles under water were examined in the eighteenth century they were found in some cases to have become petrified to a depth of three-fourths of an inch from the surface, beyond which the timber was in its original state. Before derricks were used it is probable that piles were driven by a large maul of hard wood, which is termed by Cresy a "three-handed beetle." The block of hard wood was hooped with iron and had two handles radiating from its center, to be worked by two men, while a third man assisted in lifting it by means of a short handle opposite.

Wooden mauls are still used where sheet-piling is to be driven into a soft bottom, and heavy iron mauls or sledges are also used; but as has been frequently stated such a soft bottom should be dredged and some more elaborate apparatus used to drive the piles into a harder substratum.

The most primitive form of the pile-driving derrick is similar to the one used in 1751 by the celebrated French engineer, Perronet, at the bridge of Orleans (Fig. 32). This was arranged with a number of small ropes splayed out from the end of the lead line, so that a number of men could pull down at one time, the drop of the hammer,

* The subject of pile-driving has been restricted to the ordinary methods and operations; such unusual processes as gunpowder pile-driving and the like have not been referred to.

Pile-driving, with the assistance of the water-jet, has been described in Chapter V and in the account of the Sandy Lake coffer-dam. The ordinary operations of pile-driving, as practiced on that work, are also described in some detail.

of course, being limited by the reach of the men's arms. The windlass shown was for the purpose of raising the pile into place between the leads.

The same engineer improved upon this derrick by adding a large bull-wheel to the windlass, on which was wound a rope to be pulled by a horse from the side, as shown in Fig. 33, thus winding up the lead line on the windlass. This same apparatus is in use down to the present time, except that one seen recently had the windlass at right angles to the one illustrated.

The ram or hammer used in olden times was of oak, bound with iron, and weighed for the work at Orleans 1200 pounds for the main piles, which were 9 to 12 inches in diameter and which were driven 3 to 4 feet apart, center to center, to a depth of 6 feet into the bed

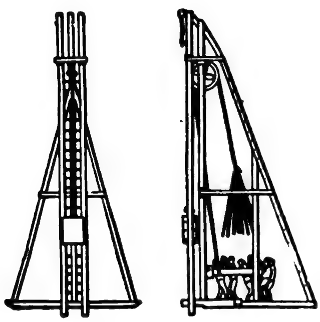


FIG. 32.—PERRONET'S PILE-DRIVER.

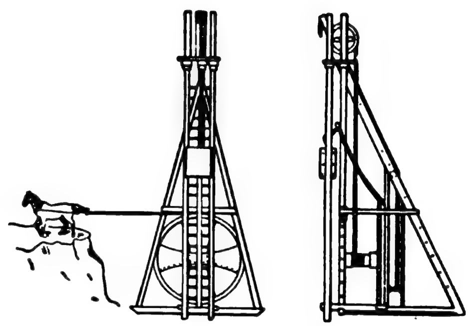


FIG. 33.—PERRONET'S BULL-WHEEL PILE-DRIVER.

of the river; the ram for the sheet-piles only weighed half as much, the sheet-piles being about 12 inches wide by 4 inches thick.

At the bridge of Saumur, which was built about the year 1756, De Cessart employed a driver with a bull-wheel, in the periphery of which were set pins, to form handles for the men to pull upon and rotate the wheel. Eight men, by making three turns of the wheel, raised the ram weighing 1500 pounds 6 feet, when it was unhooked and allowed to drop. The piles cost from two to five dollars each in place.

A very simple form of pile-driver is shown in Fig. 34, and was described in the *Engineering News* of March 16, 1893, by Julian A. Hall. The hammer is hewed out of a section of a hardwood log, and has pieces bolted on the sides to hold it in the leads, which should give plenty of clearance. The derrick was constructed of very light timber, the verticals being 4-inch sawed stuff and the bottom tim-

bers 6×6 inches. The rope passes over the sheave *A* and down over the tops of the steps *B*, *B*, on which the men stand to pull the line and thus operate the hammer. This was a very inexpensive apparatus and was found to work well. Where there is already in use a heavier hammer of cast iron it can be used by striking light blows. The construction of the ordinary pile-driver derrick is a simple piece of framing, when good straight timber is easily obtained, the essential features being to keep the leads free from any obstruction for the hammer and to have efficient bracing.

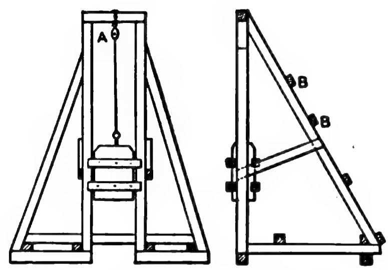


FIG. 34.—SHEET-PILE DRIVER.

For bracing a derrick under 25 feet a straight-back brace or ladder having two horizontals running to the leads, and two side-braces will be sufficient. But for a higher one, either additional long braces should be used or diagonals introduced between the leads and the ladder. The use of long braces is shown in Fig. 35, which is the design of pile-driver such as is used about harbors or rivers on heavy work. It would be mounted on a scow or flatboat 60 feet in length, 25 feet in width and of about 6 feet in depth. The design of smaller derricks can be approximated from this one, the bracing being used in proportion.

It will be noticed that the guides for the hammer are 4″×4″ lined with a steel plate. Two lines are provided, one being for the operation of the hammer and the other for pulling piles into place. Especial attention is called to the hooks at *A*, as these are seldom shown in the plan of a derrick and they are of constant use for clamping and guiding piles. A timber laid across is wedged tight against the pile to draw it to line, and can be used to correct a stick which is beginning to slant badly. Similar clamps of course are used on the opposite side of the leads.

Where a pile begins to sliver or split in driving, if the sliver is spiked down and the clamps used to hold it in place, the trouble can usually be corrected before the pile is badly damaged.

The use of diagonal bracing between the leads and ladder is shown in the Lidgerwood derrick (Fig. 36) in which a diagonal is introduced between each pair of horizontals. This form of bracing is very satisfactory and equally as good as the other method. The diagonals on a very large driver may be extended over two panels

and planks spiked down to the horizontals to form a platform for the workmen. In smaller derricks the diagonal bracing is most always omitted, dependence being placed in the stiffness of the leads and the bracing from the ladder and horizontals, as was done in the derrick shown in Fig. 4.

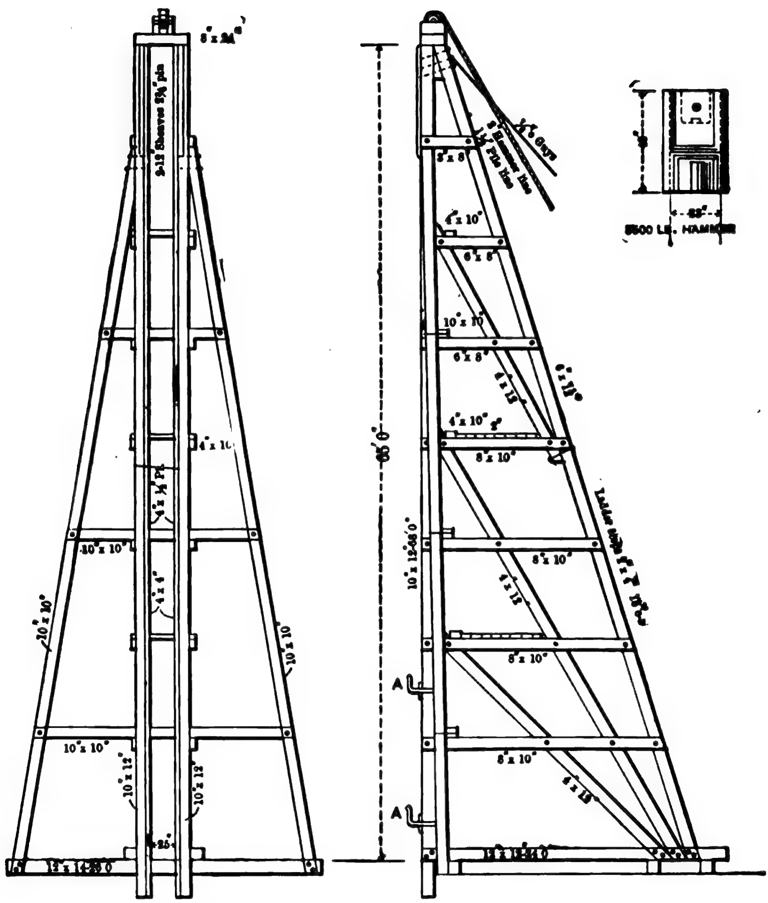


FIG. 35.—PILE-DRIVER DERRICK FOR USE ON A SCOW.

The power for driving with a small hammer weighing from 500 to 1500 pounds, may be furnished by laborers pulling, but this is a slow operation and horse-power is nearly always used where steam is not available. The power is furnished from a drum with a long lever, to which the horse is hitched and winds up the hammer by walking in a circle about the drum, the frame of which is firmly fast-

ened in place. This is called a " horse-power " apparatus and works slowly, but is a cheap and satisfactory way where a very few piles are to be driven. To the hammer-line are attached the tongs or nippers, which engage the pin in the top of the hammer (Fig. 37), and when the hammer has reached the proper height it is dropped by pulling a tripping-rope and releasing the tongs, or if the hammer is hoisted to the top of the leads, the top arms of the tongs are pushed together by the wedges on the leads and the hammer released automatically. This is a slow method on account of waiting until the tongs run down again and engage the hammer. The horse-power, of course, has a ratchet, so that the rope runs down free and usually the

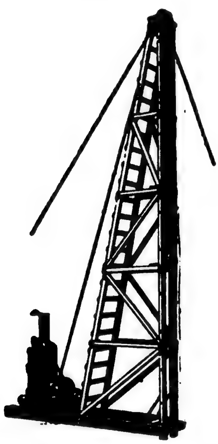


FIG. 36.—LIDGERWOOD PILE-DRIVING DERRICK.

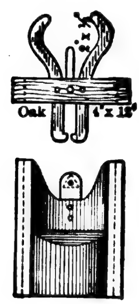


FIG. 37.—HAMMER WITH NIPPERS.

blows are hurried by overhauling the line. With the addition of a hoisting-engine all this is changed and pile-driving becomes one of the most stirring operations of the contractor. When the hammer is hoisted, the friction lever is released and the hammer descends, carrying the rope with it, as the tongs are done away with and the line attached directly to the hammer. A good engine man will catch the hammer on the rebound and materially lessen the time between the blows and likewise the cost of driving.

With a heavy hammer shorter drops are made, thus causing much less damage to the pile, which would split badly under the high drop from the use of tongs. For the smaller-sized hammers —from 1000 to 1500 pounds—an engine of 10 horse-power is mostly used, as it is usually thought best to have a surplus of power in case of need; while for a 3000-pound hammer a 20-horse-power engine

would likely prove the best and most economical, but not infrequently a 25-horse-power hoist is employed.

The cost of an outfit will vary greatly and the only satisfactory way is to get prices from responsible firms, but for preliminary estimates the cost of a 10-horse-power hoist with single cylinder and single drum may be taken at about $900, and for a 20-horse-power at $1270. Preliminary prices for other sizes of single-cylinder, single-drum hoists, may be obtained from the formula:

$$\text{Cost} = \sqrt{81{,}000 \times \text{horse-power}}. \left(\begin{array}{c}\text{Prices for 1920 from}\\ \text{60\% to 90\% higher}\end{array}\right)$$

The double-cylinder engines will cost about 10 per cent. more and double drums about 10 per cent. additional to this.

Pile-driver derricks will vary much in cost owing to the location, on account of the cost of timber, but a minimum cost for a first-class derrick will be $6 per vertical foot and a maximum of $8. Being such a simple structure the easiest and safest way will be to make an estimate for each case.

In the selection of an engine it is well to remember that with a double drum a second pile may be hoisted into place, while the first one is being driven, as all derricks are, or should be, provided with two sheave wheels at the top for this purpose. While a single-drum engine has a spool for this purpose, it cannot be used very satisfactorily.

A pile-driver on a scow is shown in Fig. 38, such as was used in driving piles on the New York State canals. Another pile is just being hoisted into position. The hoisting-engine has no protection, but a shed or house is mostly provided as a protection from the weather.

While little change has ever been effected in the design of pile-driving derricks, the adoption of steam-hoists was a great improvement, as was also the invention of the steam pile-hammer by James Nasmyth. The principle is the same as that of steam forging-hammers, and was applied by Nasmyth to pile-driving in 1845, the hammers of this class bearing his name to-day. His idea was that the drop-hammer was calculated more for destruction than for useful effect and he termed it the " artillery or cannon-ball principle." Besides this the action of the drop-hammer even with the use of the " monkey " engine was somewhat slow.

Samuel Smiles says that " in Nasmyth's new and beautiful machine he applied the elastic force of steam in raising the ram or driving-block, on which, the driving-block being disengaged, its

whole weight of three tons descended on the head of the pile, and the process being repeated eighty times in a minute the pile was sent home with a rapidity that was quite marvelous as compared with the old method. In forming coffer-dams for piers and abutments of bridges, quays, and harbors, and in piling the foundations of all kinds of masonry the steam pile-driver was found of invaluable use by the engineer. At the first experiment made with the machine Mr. Nasmyth drove a 14-inch pile 15 feet into hard ground at the rate of sixty-five blows per minute. The saving of time effected

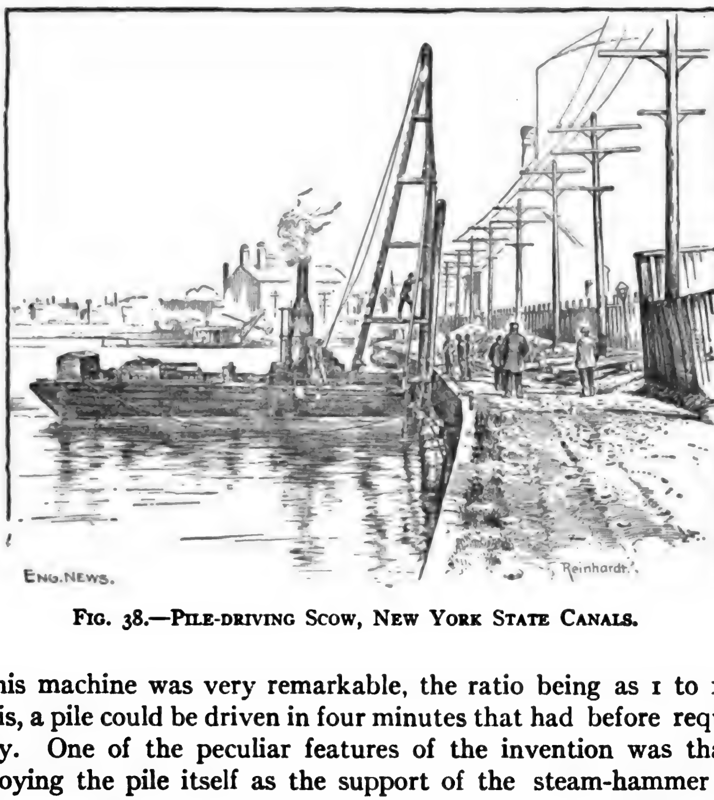


FIG. 38.—PILE-DRIVING SCOW, NEW YORK STATE CANALS.

by this machine was very remarkable, the ratio being as 1 to 1800; that is, a pile could be driven in four minutes that had before required a day. One of the peculiar features of the invention was that of employing the pile itself as the support of the steam-hammer part of the apparatus while it was being driven, so that the pile had the percussive force of the dead weight of the hammer as well as the lively blows to induce it to sink into the ground. One of the most ingenious contrivances of the pile-driver was the use of steam as a buffer in the upper part of the cylinder, which had the effect of a recoil spring and greatly enhanced the effect of the downward blow."

Many modifications of this hammer have been manufactured, and one much used at present is the Warrington-Nasmyth hammer, made by the Vulcan Iron Works. This hammer (Fig. 39) is made in three sizes, the weight of the striking parts being 550 pounds for sheet-pile work, 3000 pounds for medium pile work, and 4800 pounds for use on heavy work. This machine is provided with a positive valve-gear, a short steam passage to avoid the waste of steam, a wide exhaust opening to prevent back pressure as the hammer drops, a piston-head forged on the rod, and channel bars on the side to allow the pile to be driven lower than the leads of the derrick. The hammer is attached to the hoist rope, but this is left slack when the hammer is resting on the head of the pile, steam is turned on and the hammer pounds automatically at the rate of sixty to seventy blows per minute until the pile is driven. The bottom casting which rests on the pile is a bonnet which encases the top and prevents brooming or splitting.

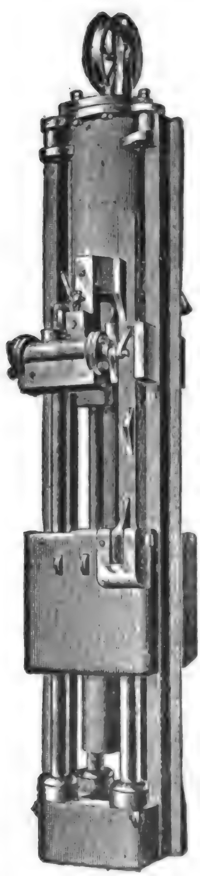


FIG. 39.—WARRINGTON-NASMYTH STEAM PILE-HAMMER.

The hammer should have plenty of play in the leads, and the steam-pipe should extend half way up the derrick to save length of hose. This hammer has a record of as high as seventy-five to one hundred piles per day, and one account gives the record of 3000 lineal feet of piling per day at a cost of $50, the number of men employed being sixteen and the coal consumption one ton. This hammer is shown in Fig. 40 in use driving piles for bridge work on the Fair Haven bridge.

Another form of the Nasmyth hammer is the Cram (Fig. 41) which is very simple in construction. The driving-head is hollow and the steam enters through a hollow piston-rod, causing the head or cylinder to rise on the rod. Four sizes are made, with hammers of 430 pounds, 2000 pounds, 3000 pounds, and 5500 pounds. The small hammer, which is listed at $300, is used for sheet-pile work by inserting a "follower" of oak which fits the base or pile cap, and which has a slit in the lower end to fit the sheet-pile. The number of

blows per minute is the same as other steam pile-hammers and an average of eighty-three piles per day of ten hours is reported, where they were driven 17 feet into sand and oyster-shells in the Passaic River, the largest day's work being 121 piles, or nearly double the best work with an ordinary hammer.

There are a number of new types of steam pile-hammers on the market, one of the best of them being the Arnott, manufactured by

FIG. 40.—WARRINGTON-NASMYTH HAMMER, FAIR HAVEN BRIDGE.

the Union Iron Works of Hoboken, N. J. As may be seen from Table I, either steam or compressed air can be used, as may be most convenient, but the size boiler given would be too small if jetting pumps are to be used; the amount of horse-power of boilers to be added for this may be found in Chapter V on Jetting Piles. It must be borne in mind, however, that a steam hammer will keep the pile moving all the time and in many cases accomplish the same results as jetting, and make it unnecessary to provide a jetting plant.

TABLE I.—ARNOTT STEAM HAMMERS.

Size Number.	Total Weight Pounds.	Weight of Ram. Pounds.	Dimensions Over All. Height × Width × Depth Inches.	Cylinder Diam. × Stroke Inches.	Total Downward Force Steam Plus Ram Pounds.	Number of Strokes per Minute.	Power in Foot-pounds per Minute.	Steam Boiler H.P. Req. at 80 lbs. Pressure per Sq. Inch.	Comp. Free Air per Minute at 80 lbs. Pressure per Sq. Inch Cu. Feet.	Suitable for	Size of Hose Inches.
0	12,100	2,550	118×28×20	10½×24	7,800	100	1,562,000	50	750	Large concrete piles, steel pipe and sheeting, extra large round and squared piles and sheeting.	2
1	8,000	1,548	94×28×18	9½×21	5,800	110	1,117,000	30	600	Medium concrete piles, heavy steel sheeting and pipe, large round and squared wood piles and sheeting.	1½
2	5,500	890	81×25×15	7¼×16	3,300	130	577,700	18	300	General work, small concrete piles, medium round or squared wood piles and sheeting, ordinary steel sheeting.	1¼
3	4,500	663	74×23×13	6¼×14	2,470	135	445,200	15	200	Minor work, small round or squared wood piles, 3″ to 6″ wood sheeting and light steel sheeting.	1¼
4	2,500	363	60×20×11	5¼×12	1,683	150	252,450	10	150	Light work, light round or squared wood piles, 2″ to 6″ wood sheeting and light steel sheeting.	1
5	1,400	214	47×17×9	4¼×9	1,085	200	162,750	8	100	2″ to 4″ wood sheeting and very light steel sheeting.	1
6	850	129	40×14×8	3¼×7	636	250	93,180	5	60	2″ to 3″ wood sheeting.	¾
7	365	70	31×10×6	2⅜×5	364	300	45,000	3	40	1″ and 2″ wood sheeting.	¾

Efficiency is computed on 60 lbs. mean effective pressure at cylinder.

The Arnott hammer uses the pressure of the steam in striking the blow and on account of the large number of blows per minute,

FIG. 41.—CRAM-NASMYTH STEAM PILE-HAMMER.

the result of the use of this hammer is greater in foot pounds per minute than almost any other hammer on the market.

The hammer, fitted with a wood-cushioned head for driving concrete piles, is shown in Fig. 42, and for such use it should be

either the No. 0 or the No. 1 hammer. The No. 1 hammer is also the best for general driving, although for smaller wood piles the No. 2 hammer will give good results.

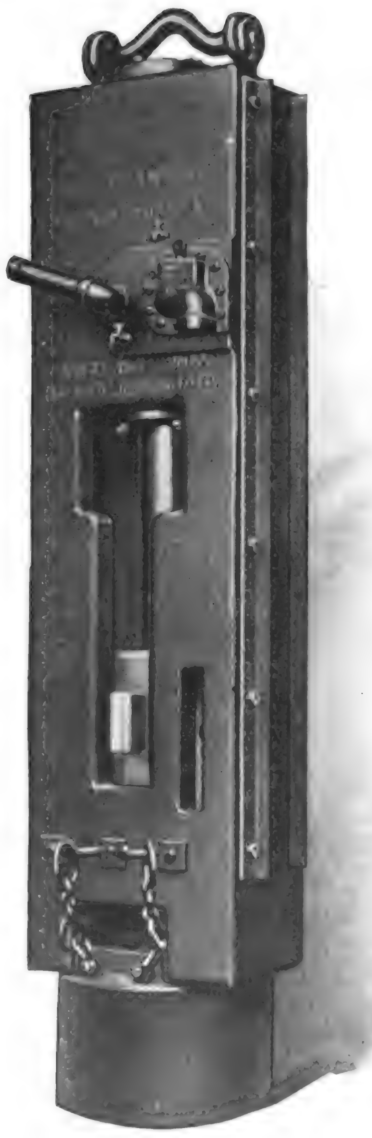


FIG. 42.—ARNOTT-NASMYTH STEAM HAMMER.

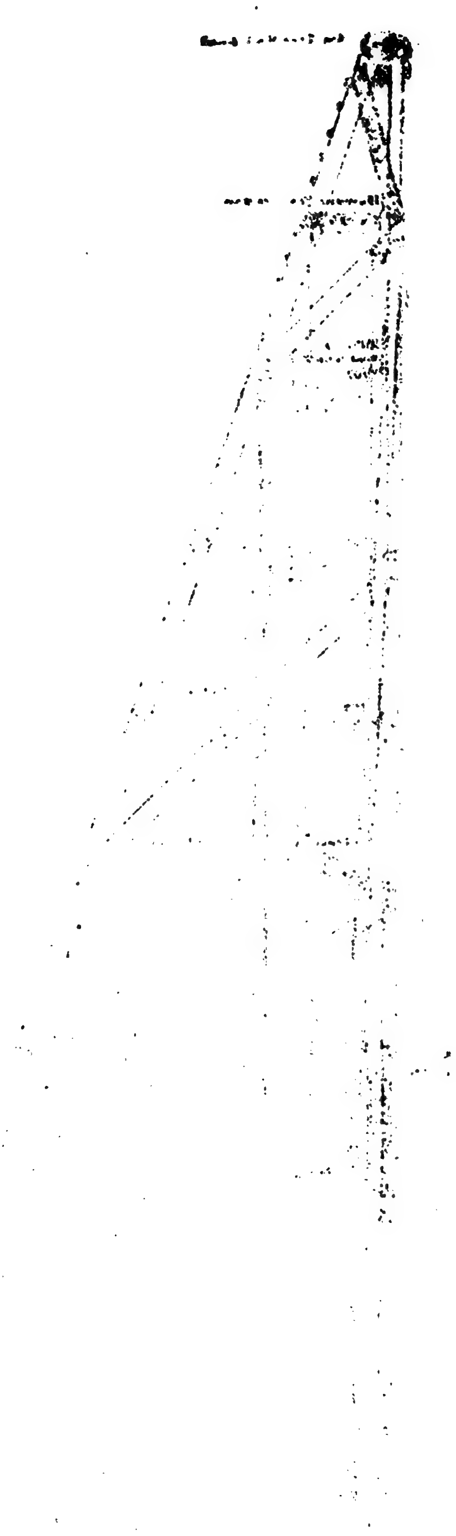
The guides for the leads are shown in Fig. 43, although the dimensions given in Table II may be slightly varied.

"All standard stock hammers are provided with heavy angle iron guides, attached with hexagon head tap bolts as per data below. One of the many advantages of detachable guides is that the hammer can be placed in standing leads by removing the rear guide tap bolts, backing hammer in place and replacing guides, thus obviating the necessity of digging pits, etc., for entering the hammer if they are not removable.

"Note that dimension A is actual width of hammers and cannot be altered, but space B between angles can be slightly increased as far as permissible and can be decreased as may be desired.

"Unless otherwise directed guides will be attached as per B."

The floating drivers used by the author have usually been on scows $20 \times 60 \times 4$ feet deep, so that they would be of use on bents closely spaced or in confined locations, and sometimes they have been as narrow as 18 feet. The depth must



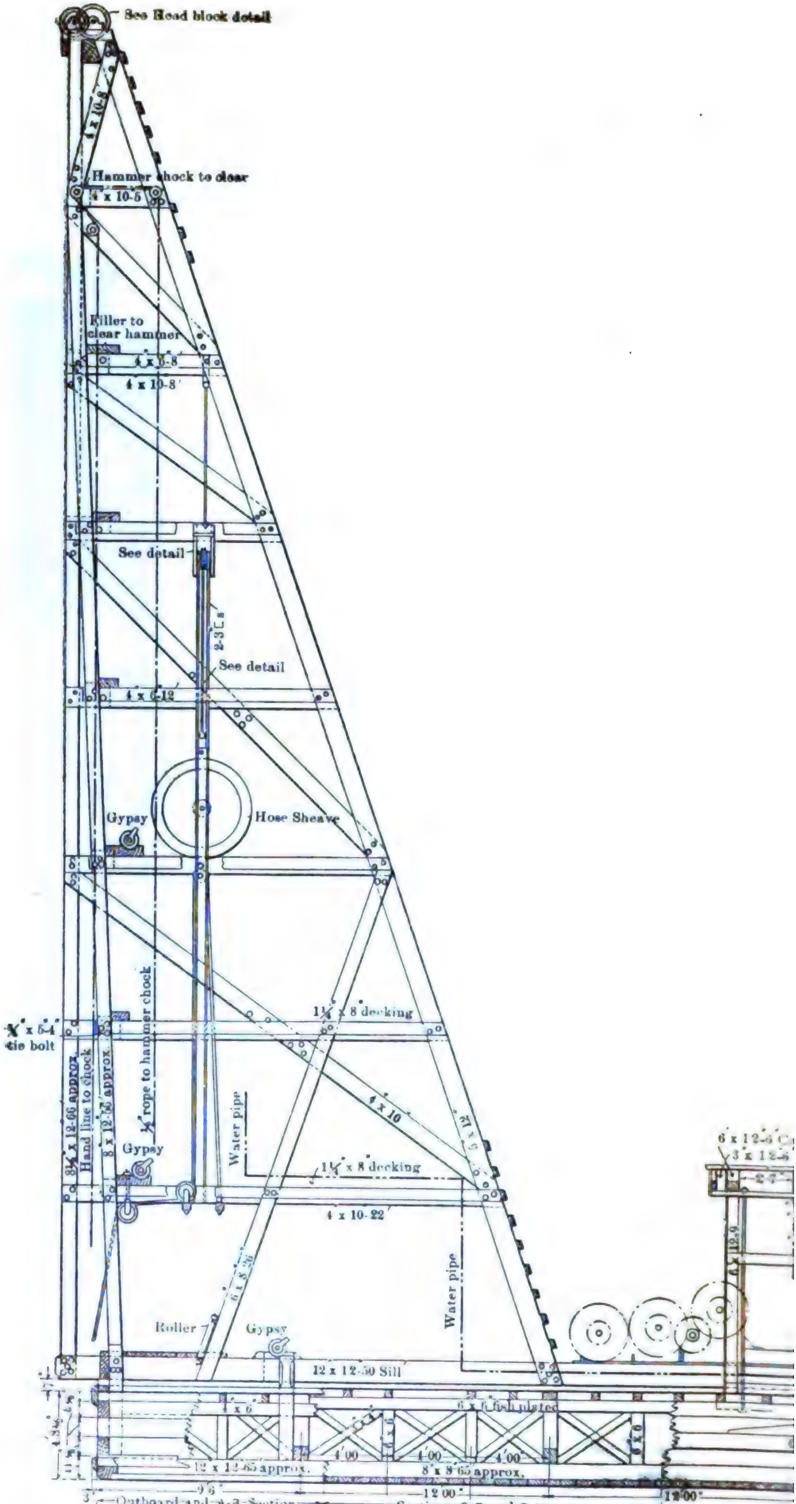
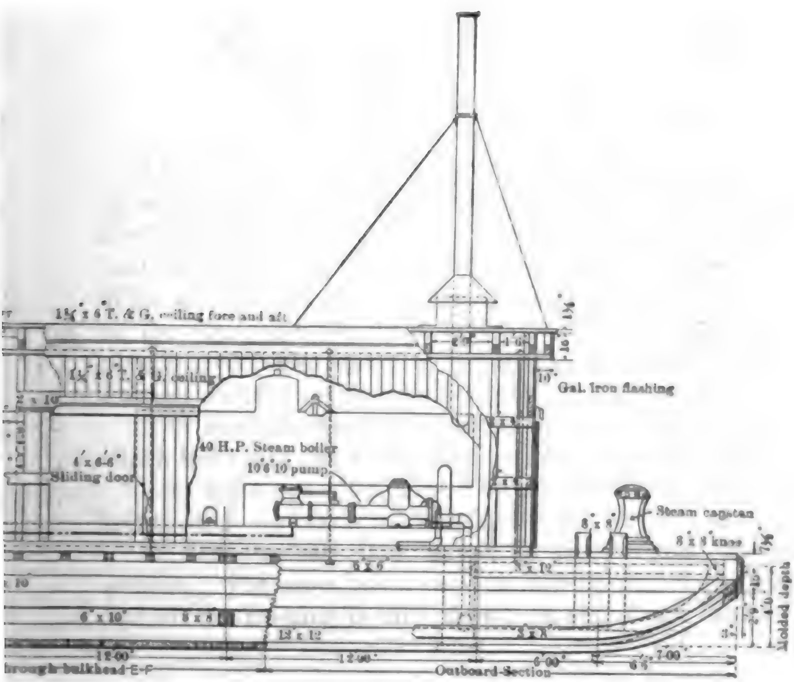


FIG. 44.—UNITED STA



LOATING PILE DRIVER. *(To face page 67.)*

usually be kept down to 4 feet to allow of use in shallow water, and for working close in on beaches on the tides.

TABLE II.—ARNOTT STEAM HAMMER GUIDES.

Size.	0	1	2	3	4	5	6	7
Dimension A.......	28″	28″	25″	23″	20″	17″	14″	10″
Dimension B.......	$8\frac{1}{2}$″	$8\frac{1}{2}$″	$6\frac{1}{2}$″	$5\frac{1}{2}$″	$4\frac{1}{2}$″	$4\frac{1}{2}$″	$3\frac{3}{8}$″	$3\frac{3}{8}$″
Dimension C.......	8″	8″	6″	5″	4″	4″	3″	3″

The complete plans of a floating driver recently constructed at Portland, Oregon, under J. F. McIndoe, Major, Corps of Engineers, U. S. A., for use on Government work, Figs. 44, 45, 46, 47, and 48, show a first-class piece of plant of this character.

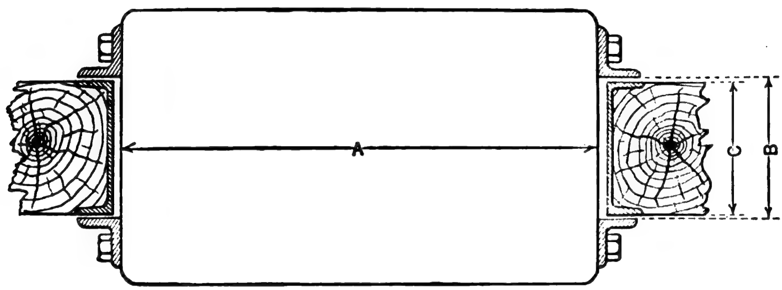


FIG. 43.—ARNOTT STEAM HAMMER GUIDES.

The scow is somewhat larger than ordinary, 24×70 feet$\times4$ feet $7\frac{1}{2}$ inches deep, and is not provided with a fresh-water tank built into the hull, as is necessary for a driver to be used on salt water. The bottom planking is 4×10 inches and the deck planking $3\frac{1}{2}\times6$ inches. Wide planking is usually fastened with three $\frac{3}{8}\times7$-inch boat spikes at the ends and two boat spikes at intermediate points. Narrow planking with two spikes at the ends and one at intermediate points. For salt water the spikes and all other fastenings should be galvanized.

The side planking or strakes should be through-fastened with clinch-bolts and rings and edge-fastened with drift-bolts.

The holes for all spikes and drift-bolts to be bored one-sixteenth inch less than the bolts or spikes.

On the outside they should be countersunk and filled in with cement. The scow has a rake or sloping end at the stern of 7 feet, and all the framing is fully shown on the plans. Scows for the best

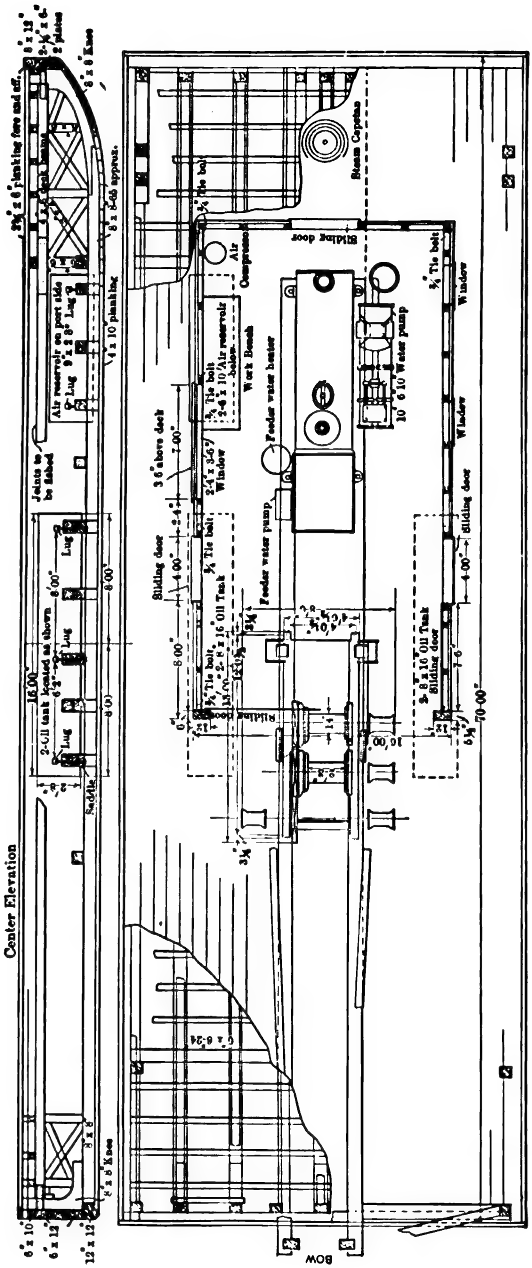


FIG. 45.—PLAN OF U. S. FLOATING DRIVER.

of work should be calked on the deck and sides with three threads of oakum, well hawsed in, and the bottom with four threads of oakum.

The seams below the water-line should all be filled with cement, after calking, and those above the water-line filled with pitch. For light or temporary work two threads of oakum may be used on the deck or sides and three threads on the bottom.

The scow is stiffened with two inside longitudinal bulkheads 6 inches thick, running full length. The leads are 66 feet high, of $8\frac{1}{2}\times 12$-inch timbers, lined with 8-inch $11\frac{1}{4}$-pound steel channels for the hammer guides. The ladder or back braces are 5×12 inches, with steps spiked on as shown. The inside braces are 6×8 inches and 4×10 inches.

Where it is necessary to tip the leads forward for driving batter piles, guys should run from the top of the leads, as shown in Fig. 35, to the stern of the scow. The head block (Fig. 48) has three sheaves, one 18 inches for the hammer line and two 16 inches for pile lines. The author's practice is to use four sheaves on the head block,

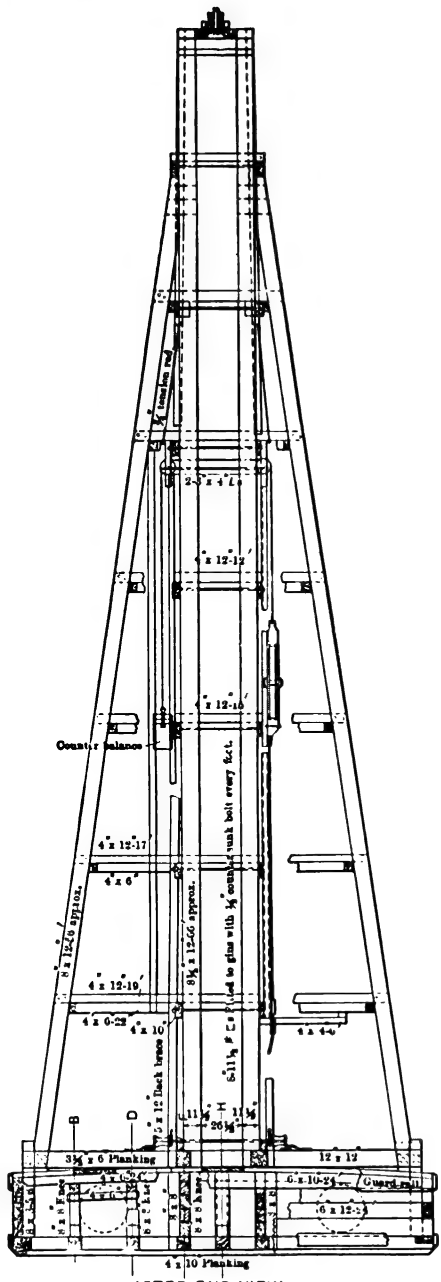


FIG. 46.—LEADS, U. S. DRIVER.

one for the hammer line, one for the pile line, and two outside ones for handling jets.

Next in importance to the scow and leads being of good design is the necessity for a first-class boiler and engine. The boiler shown in Fig. 44 is a horizontal locomotive type of 40-horse-power, or large enough to handle the jet pump for light work, in addition to operating the drop-hammer, or large enough to operate a steam pile-hammer, but not large enough to handle a hammer and two jets for heavy jetting. The usual engine for a floating-driver is a double 7×10, with the ordinary vertical boiler attached. The one shown on the plan, a double 8½×10, is provided with two spools or nigger-heads on a separate shaft to handle the bow lines coming over the gypsies. The stern lines are handled on the steam capstan shown. This driver is provided with two tanks below deck for fuel oil in

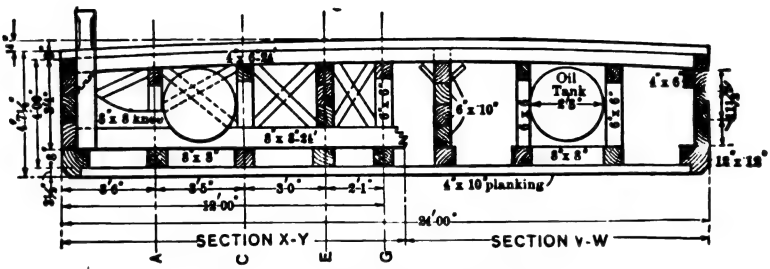


FIG. 47.—SECTION, U. S. DRIVER.

place of coal, and an air receiver to be supplied with compressed air from the air pump, for operating the wood-boring tools or such other air plant as may be found needful.

The jetting pump is a 10×6×10 duplex center-packed pump, which will deliver from 200 to 220 gallons of water per minute at a pressure of not less than 200 pounds per square inch.

The other equipment is the feed-water pump, the feed-water heater, and the fuel-oil pump. The hoist engine shown, a double-drum, double 8½×10, would be heavy enough for a large steam-hammer and for piles a hundred feet or more in length.

The scow should be protected originally by two coats of copper paint below the water line, the bottom covered with tarred ship felt and 2-inch plank sheathing, which should also have two coats of copper paint. For protection against teredo, this copper paint should be renewed about every six months. The remainder of the woodwork should be painted with two coats of the best lead

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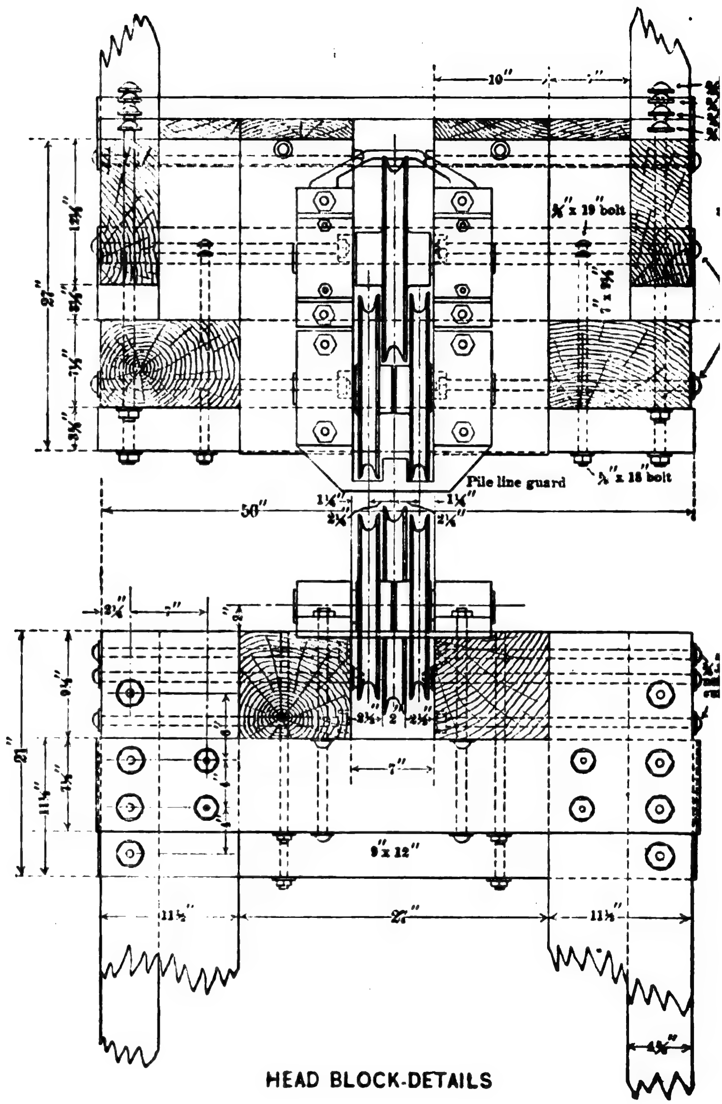
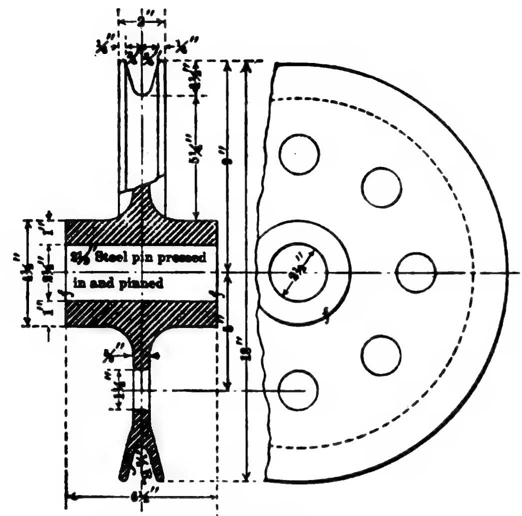
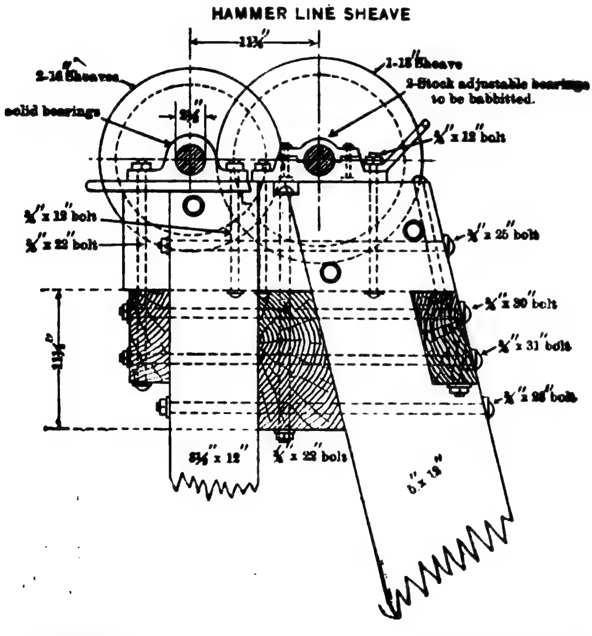


FIG. 48.—HEAD BL



HAMMER LINE SHEAVE



paint of pleasing color. The deck should be kept covered with 1-inch boards as a protection from the calks in the soles of the workmen's shoes.

The details of the jetting pipes, hose, and jets will be found in Chapter V on Jetting Piles.

The specifications for the government floating driver are given in full in Appendix VII.

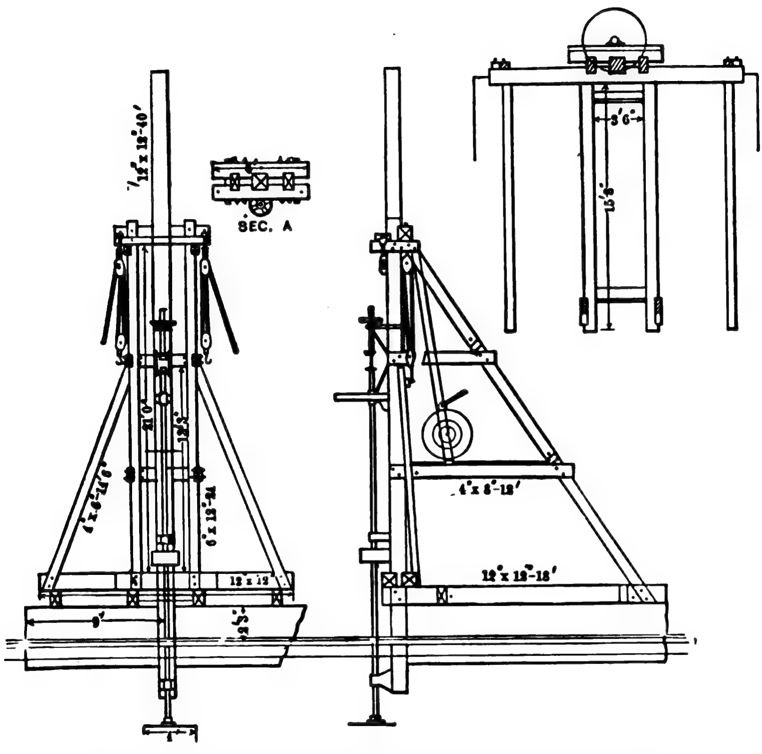


FIG. 49.—MACHINE FOR SAWING OFF PILES UNDER WATER.

Mention has been made of the use of a rock drill as a Nasmyth hammer, on the Great Kanawha River coffer-dams; and where any amount of driving is to be done it will certainly be wise to use a hammer of the Nasmyth type.

The guide-piles of a coffer-dam should always be driven with the idea of using them as a support for pumps, engines, derricks, and the like, although it will often be found cheaper to rig up on flatboats when there is danger from floods. In determining what load a pile will carry from this source, or when driven as a founda-

tion pile to support the masonry, Wellington's formula is at once the most accurate and the easiest to remember and use. For a drop-hammer, multiply twice the weight of the hammer in pounds by the drop in feet and divide by the last sinking in inches plus one, and the result is the load in pounds the pile will carry, with a factor of six for safety. This is easily remembered as $2wh$ over $s+1$, and is always ready for use. For the steam-hammer the form is $2wh$ over $s+0.1$, the "wh" representing the dynamic effect of the hammer. See Appendix XI, Piles and Pile Driving.

Where piles have been firmly driven and they are to be removed when the work is done they can be cut off under the water by a machine similar to Fig. 49, which can be operated from a barge. The description in the *Engineering News* gives but little information in addition to the drawing. The shaft works in cast-iron sleeves

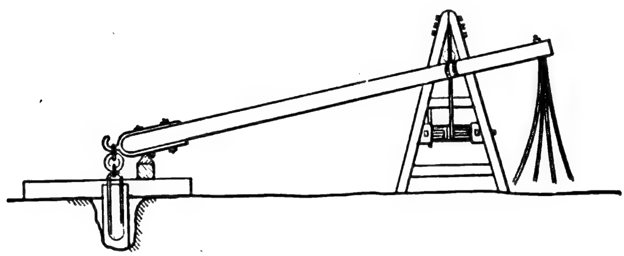


FIG. 50.—PILE-PULLING LEVER. AFTER CRESY.

attached to a timber, which slides in the leads, this being operated by the winch shown in side elevation. The final adjustment is made by the hand-wheel on the 3-foot adjusting screw. Where the piles are not so solidly driven they can be pulled out with a lever, an old form of which is given by Cresy (Fig. 50). In place of the pin and links, a chain closely wrapped around the top of the pile is usually made use of.

The apparatus used on the New York State canal work (Fig. 51) consisted of a strong frame mounted on a scow, from which was suspended a heavy set of falls to attach to the chain wrapped around the head of the pile. The pulling was done by an engine placed on the scow.

The construction of coffer-dams with sheet-piling has led to the use of a number of forms of sheet-piles, some of which are driven only as a protection to the puddle, while others are nearly or quite water-tight in themselves. The principal forms are shown in Fig. 52, the simplest form being plank of some considerable thickness

(*a*), for which Stevenson specified $4\frac{1}{2}$ inches by not exceeding 9 inches in width for the Hutcheson Bridge. The points are sharpened as at (*i*) so they will draw together in driving, and as at (*j*), to cause them to drive straight and easy. The same principle is embodied in the patent metal point shown at (*k*), which is used to protect the point when driving through coarse gravel.

The piles at Buda Pesth were increased to 15 inches square in order to resist the pressure brought upon the sides of the dam by the puddle, the water, and also by the ice. Flat plank are also used by driving two or more rows as at (*b*), the second and third

FIG. 51.—PILE-PULLING SCOW, NEW YORK STATE CANALS.

rows being used to close the cracks in the main row of piles and retain the puddle. An example of this will be given in the next article, where it was used on the Michigan Central Railway. The extra rows may be of thinner plank if they can be driven.

Mention has already been made, incidentally, of the use of V-shaped tongue-and-groove piling (*c*), on the Union Pacific Railway. This may be made on a beveled saw-table, the saw cutting half through the plank from opposite sides at each cut. This will produce a reasonably tight wall, if care is used in driving and if the points are sharpened to draw them together and make tight joints.

Ordinary tongue-and groove piling (*d*) is frequently used, but a more frequent form is that shown at (*e*), like that used on the Robinson circular dam. The two pieces forming the groove and

the piece for the tongue are spiked to the 9×12 and 6-inch spikes sloping upward. A sheet-pile dam on another pier of the Arthur Kill Bridge employed piling in which the grooves were made by making two saw cuts and cleaning out between with a chisel, the tongue being formed in the same manner as at (*f*), the tongue being spiked in one side.

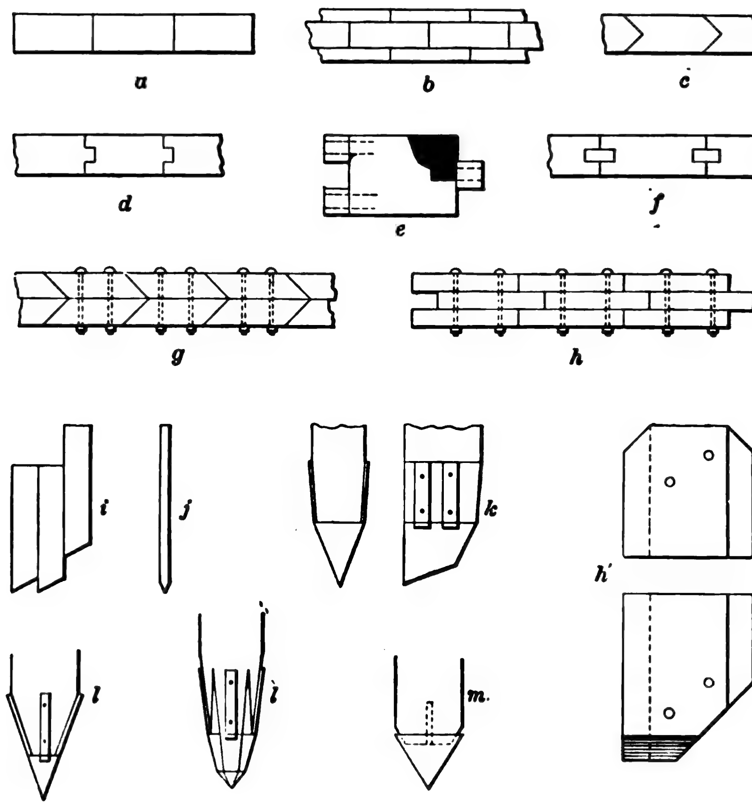


FIG. 52.—SHEET-PILES AND SHEET-PILE DETAILS.

A method which is not often employed is shown at (*f*), two grooves being made in the sheet-pile and a key driven after the piles are down. Should the piles not drive in perfect line, and the groove fail to match, the method will not be found to be a success.

Sheet-piling formed of two or more planks bolted together is being extensively used, one of them (*g*) being formed by two planks sawed with beveled edges and bolted together to form a pile similar to (*c*). This forms a pile which will drive easily on account of having

some size and which will require fewer supports in the shape of waling-pieces.

TABLE III.—PILES AS COLUMNS. SAFE LOAD PER SQUARE FOOT OF SECTIONAL AREA.

Authority.	Unit Values. Lbs. per Square Inch.		Safe Load per Sq. Ft. Lbs.
Rankine and Mahan	1000		144,000
Peronnet	786 to 990		113,184 to 142,560
Stoney	$\frac{1}{10}$ crushing wt. of dry timber.		
	Elm	1000	144,000
	Ash	860	123,840
	Beech	800	115,200
See page 109, and Appendix XI.	Spruce	650	93,600
	Cedar	610	87,840
	Oak	600	86,400
	Yellow Pine	538	77,472

Several examples already given describe the use of Wakefield patent sheet-piling (*h*), the method of sharpening being shown at (*h'*). This is constructed of three layers of plank from 1 to 4 inches thick, according to the pressure to be sustained. The center plank must be sized to keep the tongue and groove uniform, and the plank are bolted together with six bolts for a length of from 16 to 20 feet, two bolts near each end and two intermediate. For long piles, spikes should be driven between the bolts. The bolts vary from $\frac{3}{8}$ inch for 1-inch plank to $\frac{3}{4}$ inch for 4-inch plank. A coffer-dam constructed with this piling is shown in process of construction in Fig. 53, for the foundations of Charlestown Bridge near Boston. A description of this will be given in the next chapter.

Pile-shoes for use on round or square piles are shown at (*l*) and (*m*), (*l*) being patent forms. Straps of bar iron are used in many cases with success, for main piles, and sheet iron of $\frac{1}{8}$ inch thickness, bent to a "V" and spiked on, is often all that is necessary when shoes must be used on sheet-piles.

The thickness of sheet-piling should be sufficient to prevent the plank from bulging and should be calculated to stand a water pressure due to the depth, and for a span equal to the distance between the waling-timbers or other supports. This would necessitate wales every 6 feet for 3-inch plank under 5-feet head, or wales every 3 feet for a 21-feet head. Plank $4\frac{1}{2}$ inches thick would require wales every 7 feet under a 9-feet head, or every 5 feet for an 18-feet head.

Timbers 9 inches thick will carry 9 feet under a 20-feet head, while the 15-inch timbers of the Buda Pesth dam would carry 12 feet under a 33-feet head.

FIG. 53.—CHARLESTOWN BRIDGE. DRIVING WAKEFIELD SHEET-PILING.

Good timber should always be employed if it can be procured, or, if faulty stuff must be used, allowance must be made by using thicker piles and by placing the wales closer together.

The timber used for piling in various parts of the United States is Douglas fir, long-leaf pine, tamarack, redwood, cypress, and the best grades of cedar and oak. Redwood and cedar do not rot so quickly as other woods, but lack in strength for structures and for

hard driving. For temporary structures hemlock, maple, elm, and other less reliable woods, are used. The usual specification requires that piles be cut from sound, live trees, and must be free from the usual timber defects, except that a small amount of heart rot be allowed in cedar.

TABLE IV.—SAFE LOAD IN TONS ON PILES $= \frac{2wh}{s+1}$.

Weight Hammer.	Last Sinking in Inches under 15 Ft. Drop.									
	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
1200	15.0	12.9	11.3	10.0	9.0	8.2	7.5	6.9	6.4	6.0
1600	20.0	17.2	15.0	13.4	12.0	10.9	10.0	9.3	8.6	8.0
2000	25.0	21.4	18.8	16.7	15.0	13.6	12.5	11.6	10.7	10.0
2400	30.0	25.7	22.5	20.0	18.0	16.4	15.0	13.9	12.9	12.0
2800	35.0	30.0	26.3	23.4	21.0	19.1	17.5	16.2	15.0	14.0
3000	37.5	32.2	28.1	25.0	22.5	20.4	18.8	17.3	16.1	15.0
3200	40.0	34.3	30.0	26.7	24.0	21.8	20.0	18.5	17.2	16.0
3400	42.5	36.4	31.9	28.4	25.5	23.2	21.3	19.6	18.2	17.0
3600	45.0	38.5	33.8	30.0	27.0	24.6	22.5	20.8	19.3	18.0
3800	47.5	40.7	35.7	31.7	28.5	25.9	23.8	21.9	20.4	19.0
4000	50.0	42.8	37.5	33.4	30.0	27.3	25.0	23.1	21.5	20.0
4200	.....	45.0	39.4	35.0	31.5	28.6	26.3	24.2	22.5	21.0
4600	.....	49.3	43.1	38.4	34.5	31.4	28.8	26.6	24.6	23.0
5000	.....	.....	46.9	41.7	37.5	34.1	31.3	28.8	26.8	25.0

Never use loads greater than those above upper black line, except in emergency, when the lower black line may be used as the limit. In cases of rare emergency values below lower line may be used, if all conditions are certainly known. See Appendix XI.

Usually the piles are to be peeled, but often tight-bark winter-cut piles are required, as it is protection in some cases, like in salt water against teredo, where the bark will protect for a period of from one to two years, if there are no breaks or abrasions of the bark. In some particular cases the piles are required to be absolutely straight, but this is a very rigid demand and mostly they are specified to be so that a line stretched from end to end of a pile will not fall outside the stick.

The size of piles is usually specified to be not less than an 8-inch top or point and not less than a 14-inch butt. They should not run over 24-inch butts, or else they would have to be slabbed off to go in the pile-driver leads. Where piles from 70 to 120 feet

long are used the tops are often allowed as small as 7 inches, and in rare cases 5 inches or 6 inches diameter.

The driving of piles should never be carried to a point where there is danger of brooming or shattering the stick, and this must usually be left to the judgment of those in charge of the work. Fig. 54 shows the results of overdriving on the Vancouver, Wash., bridge. The best results will be obtained by using a hammer of from 3400 pounds to 4200 pounds, with a short drop. The hammer should be of a short pattern with the weight mostly concentrated

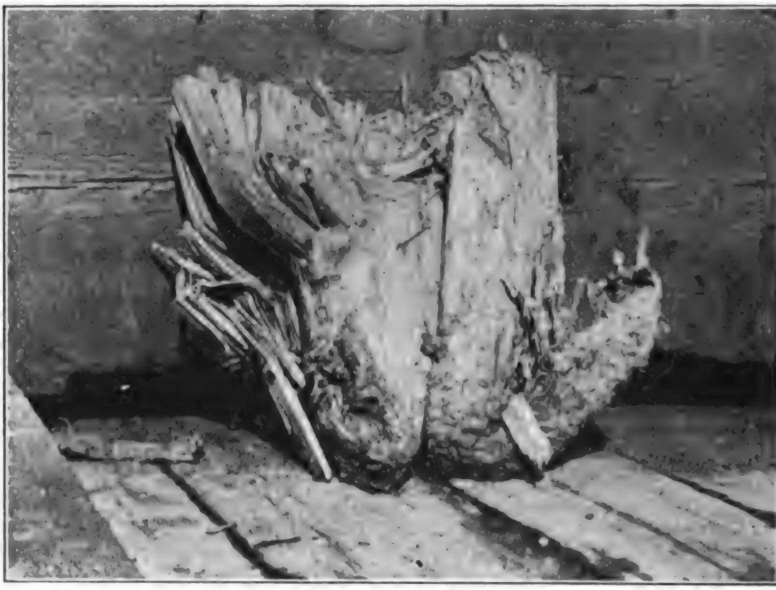


Fig. 54.—Effect of Overdriving Piles.

in the ball or bottom end. In sand, quicksand, soft clay or light gravel a steam-hammer striking from 60 to 130 blows per minute will be found more effective than a drop-hammer, and the weight should be from 5 tons upwards. The steam-hammer will usually make it possible to avoid the use of jets in this class of material, and as jetting nearly doubles the cost of pile-driving it should be avoided whenever some other method can be employed.

A recessed base on the steam-hammer acts as a cap on the head of the pile and avoids the need for pile rings, although the pile heads must be shaped up to fit the base. With the drop-hammer a follower cap, Fig. 55, may be used to avoid the use of pile rings. Where piles

are to be driven below water, the base casting on the follower, Fig. 56, can have the same shaped bonnet to fit the head of the piles, and when the piles have been followed down to the proper elevation ready for pouring the concrete around them, it will be unnecessary to cut them off under water, as the heads will be uninjured and in proper condition for carrying the load. The author has made experiments on this method above water and has had the heads of

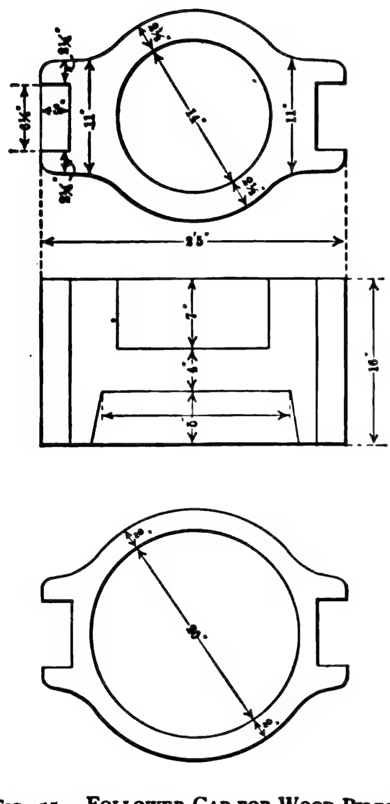


FIG. 55.—FOLLOWER CAP FOR WOOD PILES.

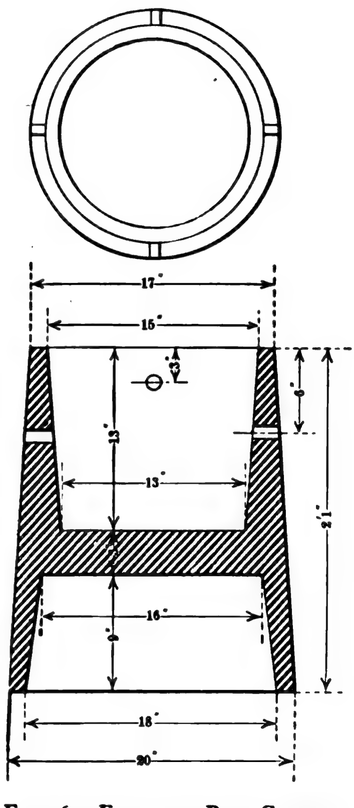


FIG. 56.—FOLLOWER BASE CASTING.

piles cut off under water by a diver after being driven by this method and found them in better condition than before, inasmuch as the fiber of the timber has been compressed and in better shape to take the bearing.

Pile rings about $\frac{7}{8} \times 2\frac{1}{2}$ inches must nearly always be used with the drop-hammer unless the driving is very soft or unless the bonnet cap is employed.

Pile shoes as shown in Fig. 52 (*l*) and (*m*) may be used in hard clay and compact gravel with good results, but in hardpan and cemented gravel there is every likelihood of the pile splitting out around the shoe and brooming up worse than if nothing was used. The triangular cast point (Fig. 57) used on some recent work in hard clay and cemented gravel worked much better, the sharp corners cutting into the material, so that a penetration of several feet was obtained. For ordinary compact material it is usually necessary only to point up the end of the pile as shown in Fig. 52, leaving off the point (*l*).

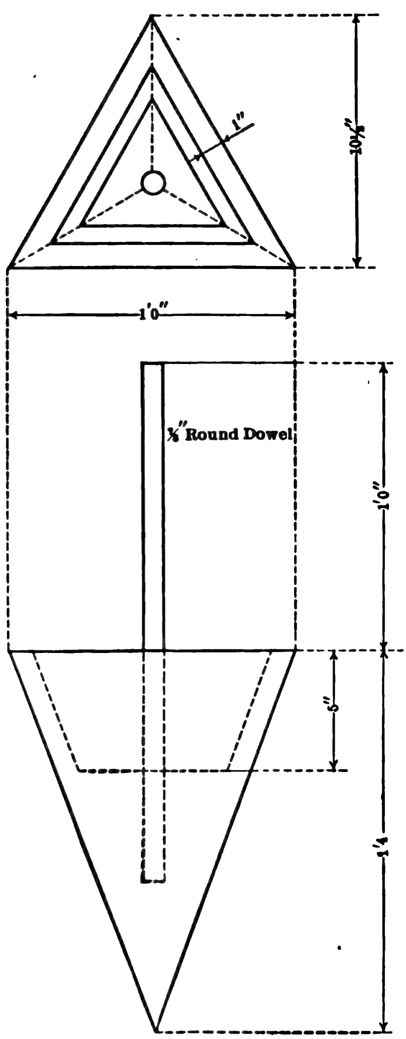


FIG. 57.—TRIANGULAR CAST PILE POINT.

Piles have been driven butt down in very soft material, to get better bearing and additional frictional area, but it is objectionable, as the small end of the pile will not stand hammering, it will spring in driving; the top is too small for proper bearing for the caps, and the part of the pile out of ground is the smallest diameter and will not carry the proper load in many cases as a timber column.

The use of swinging or pendulum leads is very often desirable where batter piles are to be driven in a trestle, and the rig shown in Fig. 58 is a very simple one to construct and very efficient. Where brace piles are to be driven under a wharf with a floating driver, the rig shown in Fig. 59 may be used, being hung on the face of the driver leads, and can be moved up and down to various elevations required to drive the braces, or to allow for rise and fall of the tide.

The ordinary crew for a land driver is six men including the foreman, and for a floating driver from seven to nine men, including the foreman and boom man. On some work an extra boom man is required and often extra men for such work as heading up the piles; for jetting work from two to four extra men are required to handle the jets. The cost of driving piles will be discussed in the chapters on Cost of Work at the end of this volume.

Pulling piles is done by using double or triple wire-rope blocks for falls on the face of the leads, the pile being gripped by a chain or

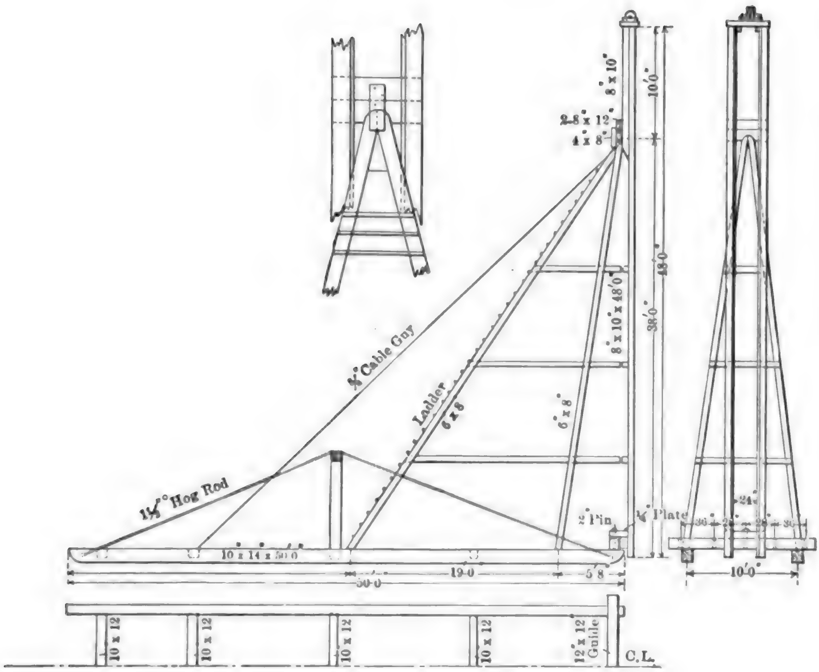


FIG. 58.—PENDULUM PILE-DRIVER LEADS.

wire-rope sling. It is usually necessary to hit the pile with the hammer in order to start it and sometimes a jet must be employed in loosening them. The pull necessary to loosen them is due partly to the friction on the pile and partly to the suction, which is overcome by taking a pull with the driver and holding on until the pile lets go. This friction and suction has been found in some cases to amount to as much as 1800 pounds per square foot on the surface of the pile, but where the friction is taken account of in the load the pile will carry, it should not be taken at over 500 pounds per square

foot in firm sand or clay and will not amount to over 120 pounds in silt.

Concrete piling have come into such general use that they can no longer be considered an experiment, and are used without much question wherever a permanent foundation is required.

They should always be used where there is any danger of wood piles decaying, that is, where piles will not constantly keep wet. Where some more permanent protection against limnora or teredo

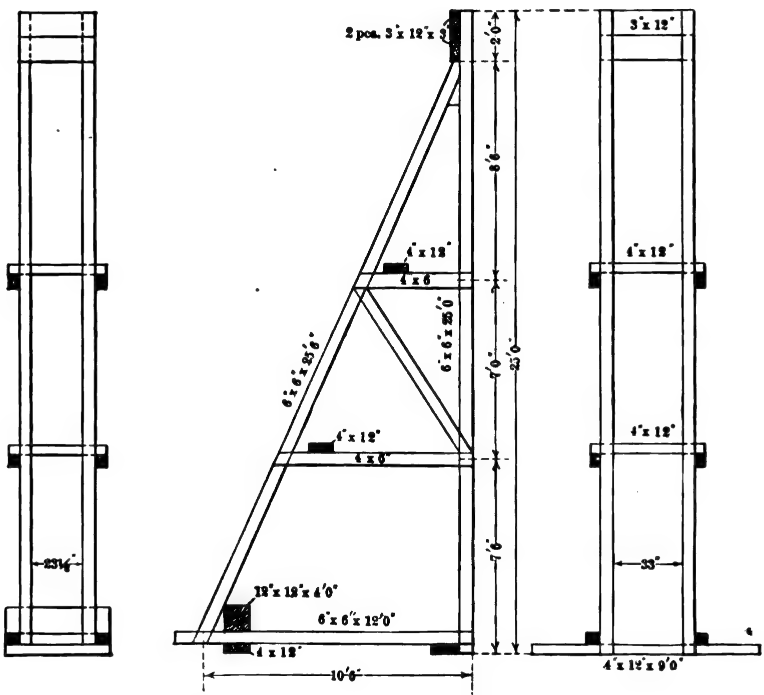


FIG. 59.—BATTER PILE-DRIVER LEADS.

than that offered by creosoting, is desired, concrete piles are very often the solution of the problem.

The Corrugated or Gilbreath pile is one of the best-known forms (Fig. 60), and the corrugations afford considerable additional frictional surface where piles are being driven into a soft bottom. This pile is a moulded pile, being usually poured into horizontal forms on the ground, the reinforcing being first placed in the form. The amount of reinforcing to be put into piles up to 35 or 40 feet is only enough for ordinary handling and driving. For the East Twenty-

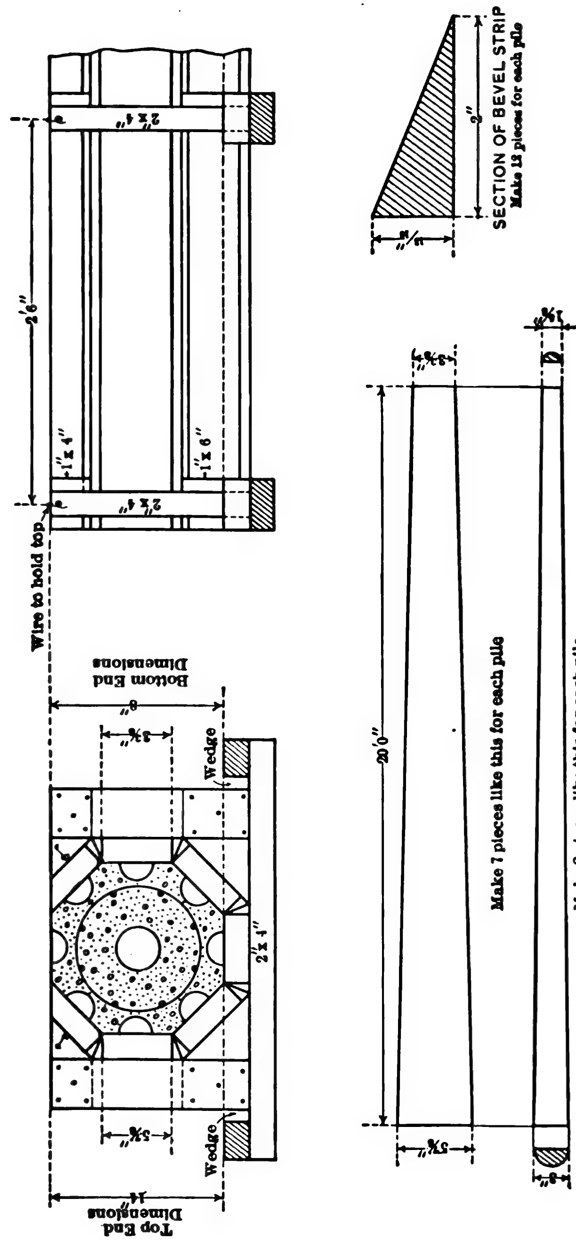


FIG. 60.—CORRUGATED CONCRETE PILE FORMS.

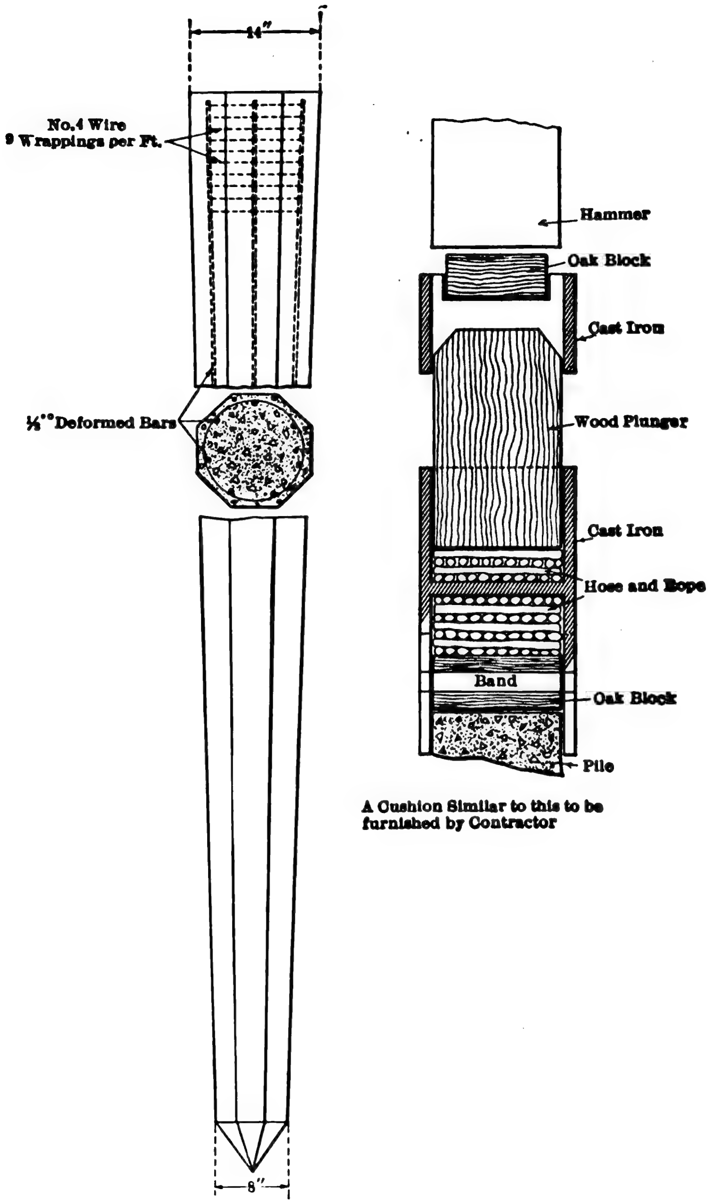


FIG. 61.—REINFORCED CONCRETE PILE AND DRIVING CAP.

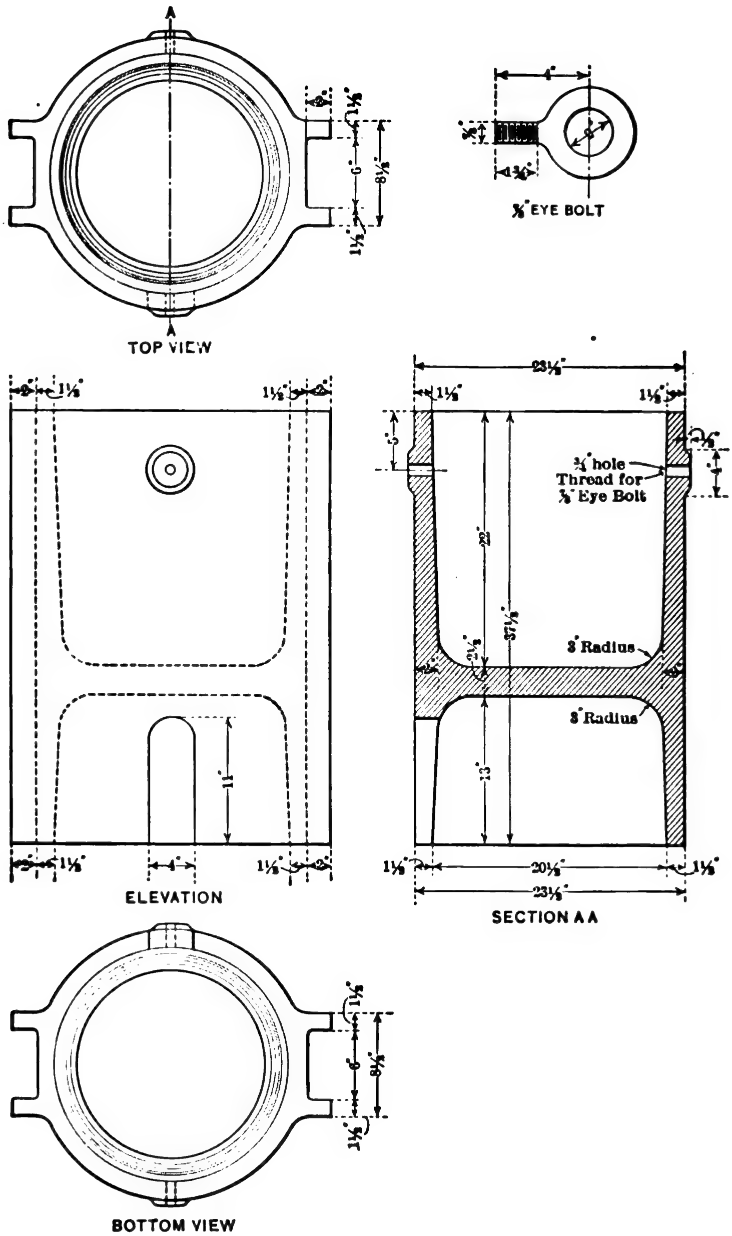


FIG. 62.—CASTING FOR CONCRETE PILE CAP.

first Street Viaduct in Portland, Oregon, the author used piles with seven half-inch round deformed bars lengthwise and nine wrappings per vertical foot of No. 4 wire (Fig. 61). The driving head specified for this is shown in Fig. 61 and is substantially that specified by the patentees of the Corrugated pile, the details of the steel casting for the cap made for this is shown in Fig. 62. The complete specifications for these piles required them to be cast in a horizontal position on the ground, of 1 : 2 : 4 concrete, and driven with a 3000-pound steam drop-hammer, and a three-foot drop. "The pile shall be allowed to set for thirty days before driving. Safe load assumed for each pile to be 22 tons, and the contractor will be required to

FIG. 63.—CONCRETE PILE CURING YARDS.

test four piles; each test pile shall be loaded with 22 tons for 48 hours and shall show no settlement. Each pile shall then be loaded with 35 tons and shall not show more than $\frac{1}{4}$-inch settlement after the load has been applied for 48 hours. Should the test piles fail to sustain the load as specified, the contractor must increase the length of the piling, so that in the opinion of the engineer the piling used will safely carry a load equivalent to 22 tons for each pile shown on the plans."

The piling used were as shown, but corrugations were added and the jet pipe cast in the center of each pile. (Fig. 60). To avoid delay, while the piling were curing in the yards (Fig. 63), test-piles of timber were driven and the ordinary formula applied to determine

their bearing capacity. The material was a sandy, slate-colored clay, moderately soft, and the test-piles were found to give satisfactory results, without the necessity of using longer ones than 30 feet. With frequent wetting down, the piles cured enough for handling

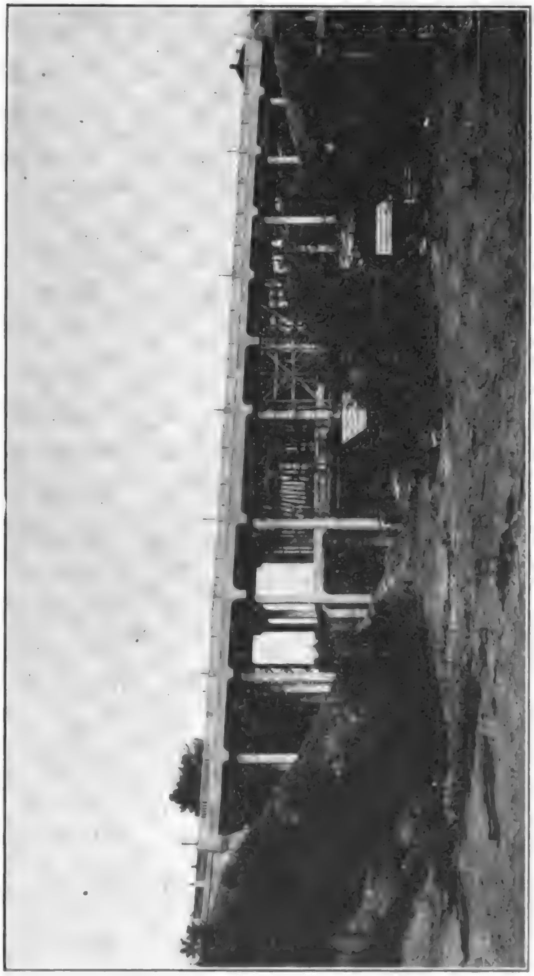


Fig. 64.—Twenty-First Street Viaduct, Portland, Ore.

without damage in about 20 days. They were driven by a 3000-pound drop-hammer, and the jet operated from a $7 \times 4\frac{1}{2} \times 10$ duplex pump, giving a pressure at the pump of about 175 pounds.

The actual driving developed the fact that the piles were not as easily damaged as wood piles, and also that for the material to be

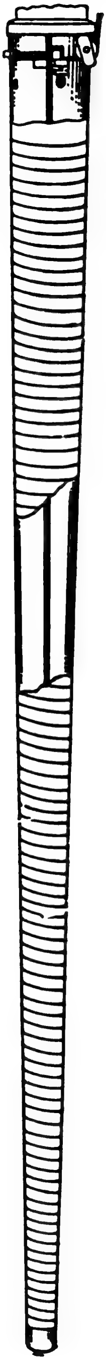


Fig. 65.—Raymond Concrete Piles.

penetrated, the center jet was not so good nor so effective as a separate jet-pipe outside the piles, and this was used for practically the entire work. The piling were made and driven under the direction of H. W. Holmes, M. Am. Soc. C. E., engineer in charge of the entire structure (Fig. 64), and also the designer.

The usual reinforcing for the corrugated pile consists of Clinton wire-cloth placed just inside the corrugations, the cloth having a mesh of 3×12 inches, the 12-inch mesh vertical, with No. 3 wire lengthwise of the piles and No. 10 wire around the inside. This reinforcement is only sufficient, as has been stated, for ordinary lengths and handling; for rough usage and for resisting bending stresses it must be increased.

There are many other forms of piles cast before driving, but the principle is the same in all of them. The piles that are built in place are of three different types, the Raymond, the Simplex, and the Clark. Using these, care must be taken not to drive piles too close to those already driven, until they have time to set, as in many instances they will be practically sheared off when the concrete is green. This may be avoided by skipping rows and then filling in later on when the concrete is fully set.

The Raymond pile (Fig. 65) has a steel shell of conical shape into which a conical steel core is placed for driving. After it is driven, the core is withdrawn and the steel shell, which is heavy enough to keep its shape, is filled with concrete. If reinforcing is used, it is placed in the shell before the concrete is poured. This usually consists of a center rod about 1½ inch round, and three ¾-inch rods near the circumference. The use of more reinforcing would assist in keeping the pile intact when driving adjacent piles.

The Simplex pile has a wrought-iron or steel-pipe form which is driven, being heavy

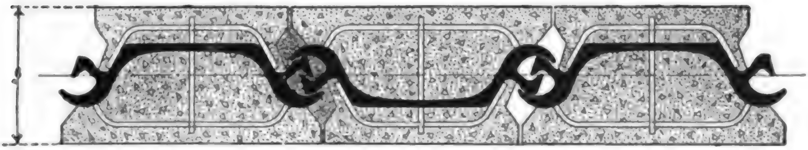
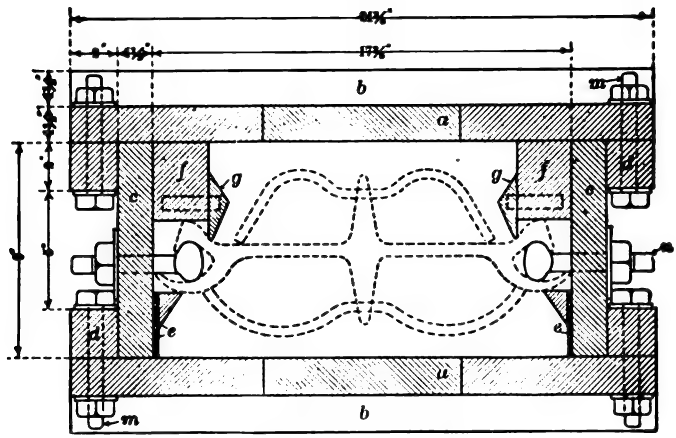


FIG. 66.—LACKAWANNA REINFORCED CONCRETE SHEET PILING.



CROSS SECTION OF FORM

FIG. 67.—LACKAWANNA SHEET PILING FORMS.

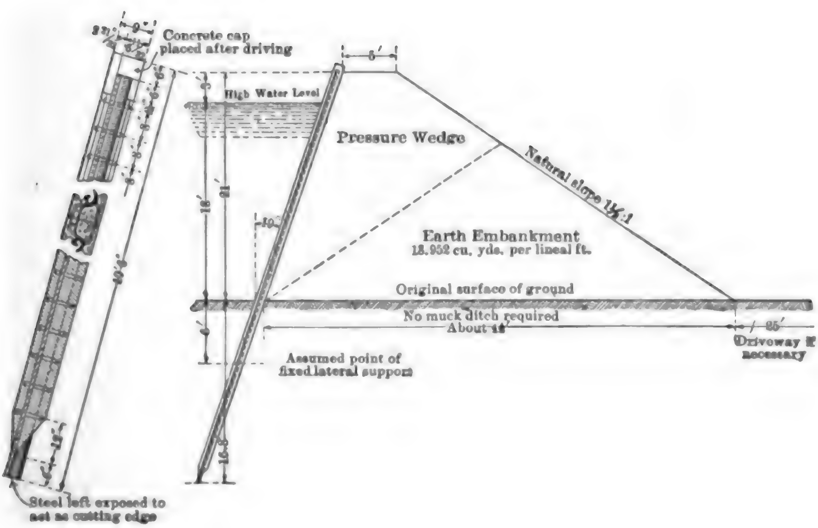


FIG. 68.—LACKAWANNA SHEET PILE LEVEE WALL.

enough to stand severe driving and it is fitted with a concrete or steel point and a hardwood driving-head. After the pipe is driven to the required depth it is filled with concrete.

The Clark pile is similar, only it has a jointed shell of $\frac{3}{8}$ inch thickness or heavier; the joint is made by an inside coupling. The pipe is driven by steam-hammer and jet-pipe. After rock or the required depth is reached, the pipe is washed out with the jet and the pipe filled with neat cement grout. Reinforcing may be placed as required.

Where sheet-piling is to be retained as part of the permanent structure, it may be made of rectangular moulded concrete piling, with reinforcing as may be necessary to sustain the load on the piling, either vertical or transverse.

The use of metal sheet-piling for this purpose, protected by concrete, is a very satisfactory solution in many cases. The Lackawanna sheet-piling used for this purpose are shown in Figs. 66, 67, and 68, the forms for moulding these piles being shown in Fig. 67, while the section of the piles is shown in Fig. 66. It is important that all surfaces of forms in contact with concrete must be smooth; preferably machine-planed. All other surfaces need be planed only enough to insure good joints. All outside surfaces may be left undressed. Stock size lumber should be used wherever possible. Forms can be extended to fit any length of piling by lengthening the top, bottom and side planks and the running strips; forms can be shortened by placing the bottom end blocks further in the form. The use of this piling in constructing a 21-foot levee is shown in Fig. 68.

The use of ordinary reinforced rectangular concrete sheet-piling is described in Volume III, where it was used on the piers in the improvement of Baltimore harbor. The cost of this class of work will be very similar to that for ordinary reinforced concrete piles, as given in Volume II, on the Cost of Foundation Work.

CHAPTER V

JETTING PILES

THE use of water jets in the sinking or driving of piles has been referred to in several places in this work, but it is the purpose of this chapter to give in proper detail the methods and plant necessary for the best results in work of this character. The first use of water jets in pile-driving was probably in 1852, at Matagorda Bay, Texas, following the idea of Gen., then Lieut., Geo. B. McClellan, Corps of Engineers, U. S. A. The nozzle was placed close to the point of the piles, being connected by an ordinary hose to a hand force pump. The method was very crude, but doubtless of much service in light material.

Hollow cast-iron piles with a disc base were sunk by a water jet from a hand pump in Chesapeake Bay in 1854, for the foundation of the Pungateague Light.

The ordinary jetting carried on for many years was only as an aid to driving with a drop-hammer and the water was usually supplied by a small duplex steam pump, about 6×4×6 in size, supplying one jet through a 2-inch pipe. This is similar to the Sandy Lake jetting described in Chapter VII, where the supply hose was only 1½ inch, the jet pipe ½ inch, attached to the sheet piles, and with a ⅜-inch nozzle.

The question of whether or not to attach the jet to the piles or to leave it loose is one about which engineers seem to differ widely, but in the author's opinion there can be but one result in the actual use, that is to leave the jet or jets loose.

With only one jet in use and that fastened to the pile with the nozzle at the point of the pile and to one side, there can be but one result—the pile will run or drive out of line. Where only one jet is used it is necessary to keep hammering the pile with short drops of the hammer, but before starting the pile, the jet should be run down into the bottom in the exact location where the pile is to be driven and to the full depth; then upon dropping the pile into the hole thus formed, it will go down under its own weight and the weight

of the hammer resting upon it, for a considerable distance or to full depth, or may then be driven in most cases to the required depth to get below scour and to carry the load, without further jetting. In case, however, it is necessary to have greater penetration than this will give, the jet can be run down the pile, first on one side and then on the other, to keep the pile going straight and to keep it properly lubricated with water its full length.

The size pump required for ordinary work may be as small as a $6\times4\times6$ or preferably a $7\frac{1}{2}\times4\frac{1}{2}\times10$, giving a pressure at the pump of from 150 pounds to 175 pounds per square inch, and supplying 140 gallons per minute through a 3-inch pipe. All pumps used in pumping salt water must be brass fitted and have brass-lined water cylinders. The supply is usually cut down somewhat by using $2\frac{1}{2}$-inch connecting and jet pipe, which should be double strength and have a nozzle made from a piece of pipe and drawn down to an orifice of from $\frac{7}{8}$ to $1\frac{1}{4}$ inch, the smaller-sized opening giving a greater cutting pressure, so that it is rarely advisable to use a nozzle larger than 1 inch diameter. This nozzle is shown in (*a*) Fig. 69, and one that works almost as good is shown at (*b*), made up of a pipe reducer and short nipple. The rose jet (*c*) and (*d*) has small holes bored around the sides to spread the water. This of course allows a greater discharge and the pump must be larger accordingly, if the main nozzle opening is kept the same size. Only a trial of this and other changes will determine which is best for any given material.

The pump shown on the floating driver (Fig. 44) is a $10\times6\times10$ center-packed plunger type, giving a supply of 220 gallons per minute at 200 pounds pressure through a 4-inch main pipe, and will give good results on two $2\frac{1}{2}$ inch jets in light material, with 1-inch nozzles, and is as large as is necessary for one jet in very compact material.

Many specifications require the use of two jets, or even three, all going at the same time in order to keep the pile going straight, and at the same time obtain the desired penetration. This will require a very much larger plant, and nothing less than a $12\times7\times10$ center-packed plunger pump will give satisfactory results for two jets operating in compact material, sand or light gravel, to considerable depths. This pump will deliver 300 gallons of water per minute through a 6-inch main pipe, at a pressure of from 175 pounds to 200 pounds per square inch. However, this kind of a plant for using two jets will add very greatly to the cost of the driving, practically doubling the cost of operating the driving plant. The pump scow shown in plan in Fig. 70 will have two boilers of locomotive type and 80 H.P. each, with fresh-water tank on salt-water work, feed pump,

injectors, feed-water heater, the jetting pump or pumps if a spare one is provided, together with the necessary pipes and valves. The

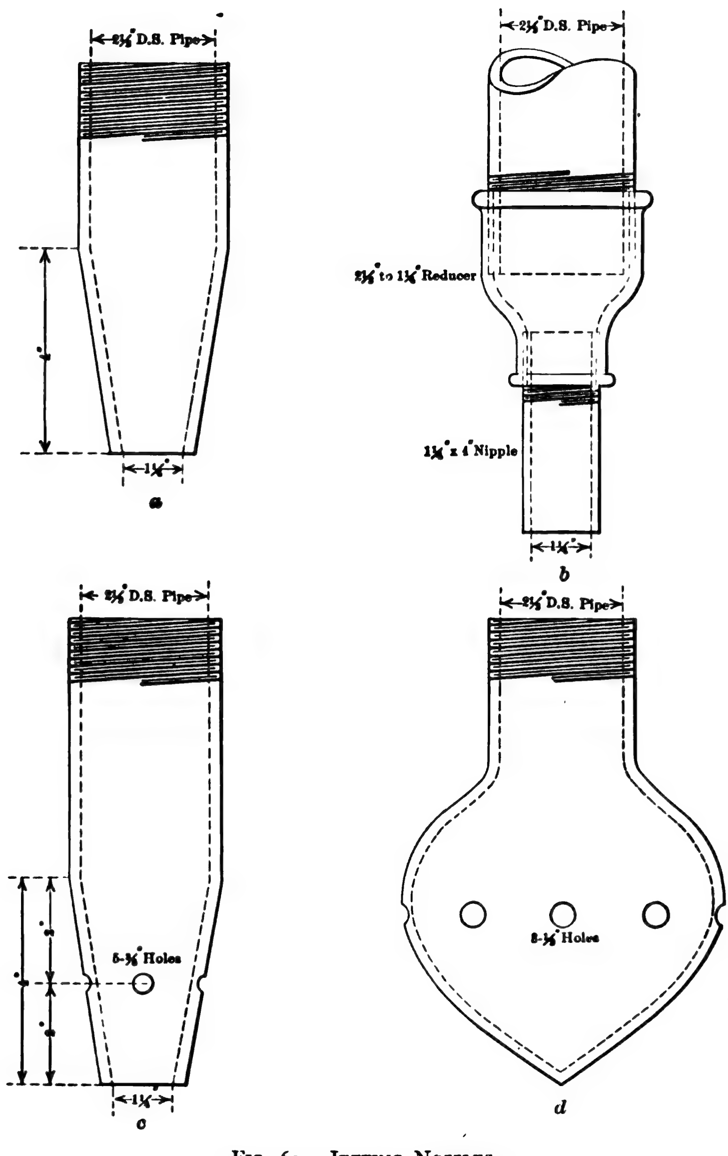


FIG. 69.—JETTING NOZZLES.

whole should be housed in by a light cover with at least 3 feet margin around the outside for walkway and handling lines. This plant,

moored alongside the driving plant, will require an extra engineer and fireman, and the driver will require two jet men to handle the jets and two nigger-head men to handle the jet lines. These extra

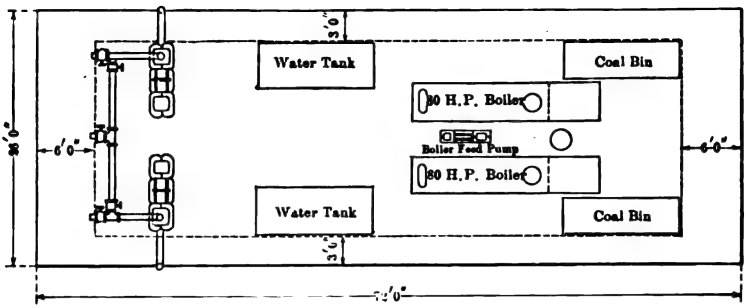


FIG. 70.—POWER SCOW FOR PUMPING.

men, with possibly a deck hand on the pump scow, together with fuel, supplies, and repairs will as stated above just about double the cost per day of operating the plant, and owing to the greater amount of plant to handle, the number of piles that can be driven in a given

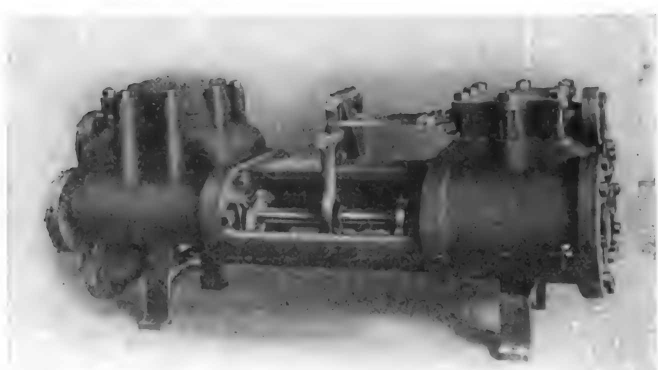


FIG. 71.—DUPLEX PISTON PUMP.

time will be greatly reduced, so that jets should only be employed where absolutely necessary.

The sizes and capacities of locomotive and internal fired boilers Figs. 74, 75, and 76 are given in Tables VIII, IX, and X, the horse-power being given on the basis of 12 square feet of heating surface per horse-power, but usually figured on the basis of 10 square feet; the sizes and capacities of pumps (Figs. 71, 72 and 73) in Tables V,

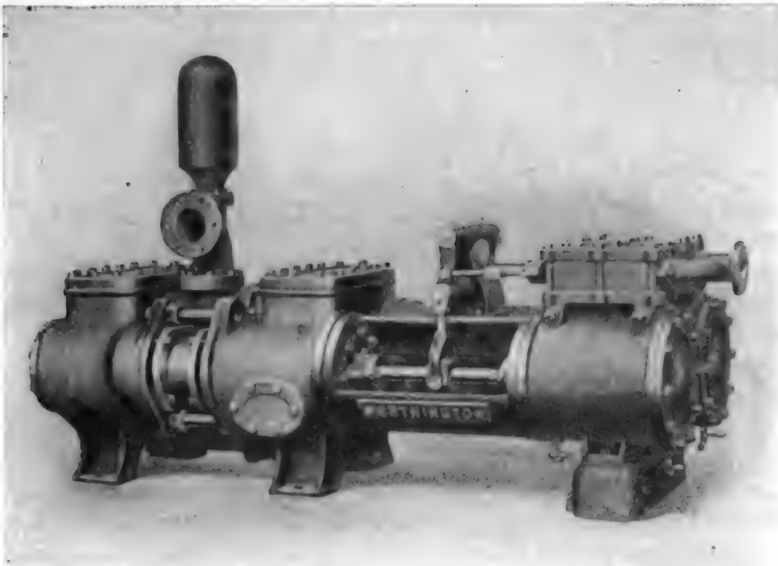


FIG. 72.—DUPLEX CENTER PACKED PUMP.

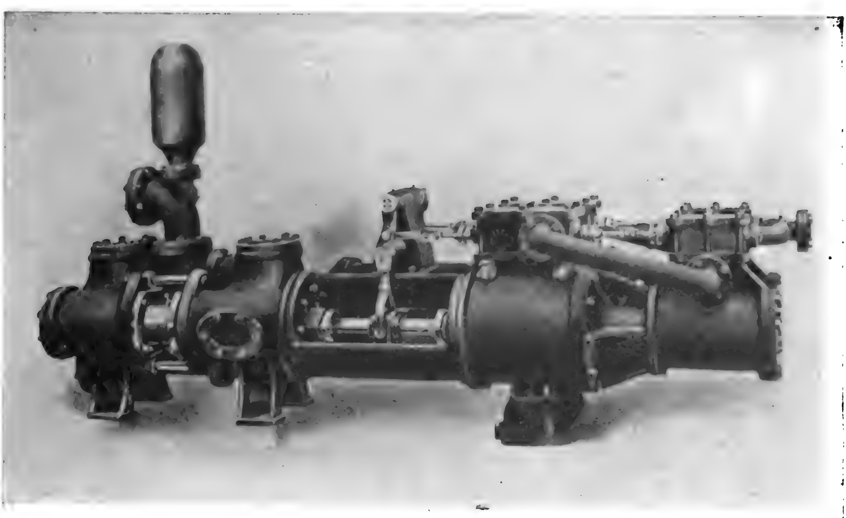


FIG. 73.—COMPOUND DUPLEX CENTER PACKED PUMP.

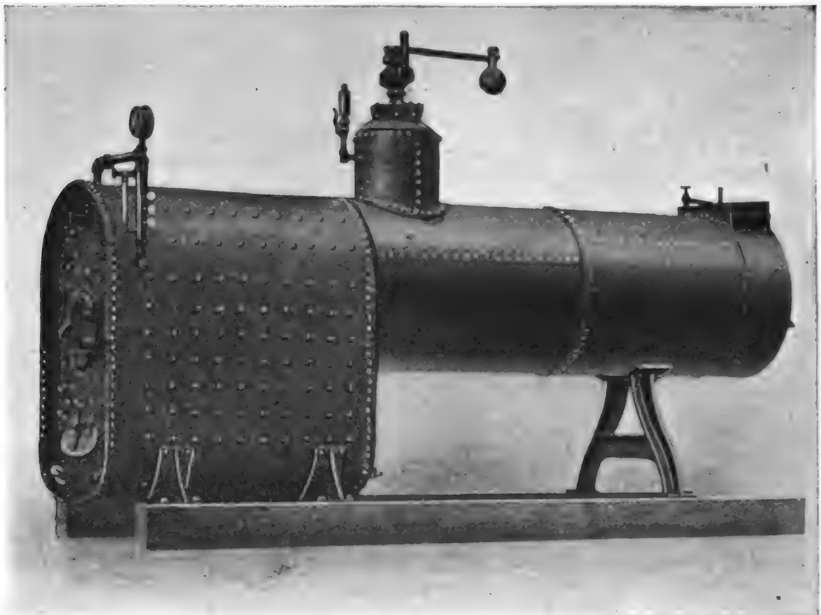


FIG. 74.—LOCOMOTIVE BOILER. WATER BOTTOM.

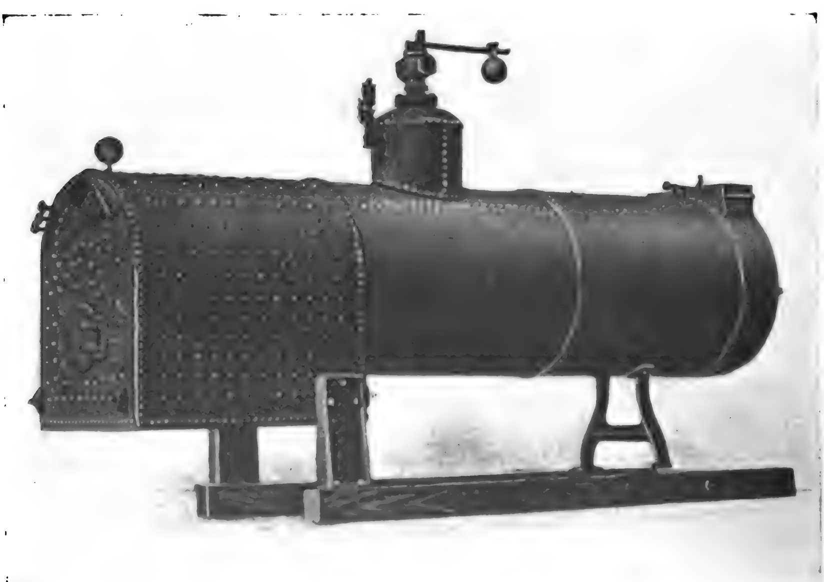


FIG. 75.—LOCOMOTIVE BOILER. OPEN BOTTOM.

VI and VII, together with the boiler recommended by pump manufacturers for each size pump, increased 50 per cent, as the ordinary duplex pump is a "steam eater," there being an almost steady stream of steam running through it.

Where very compact sand, compact gravel, or clay is to be jetted, the center-packed plunger type of pump, with compound-steam

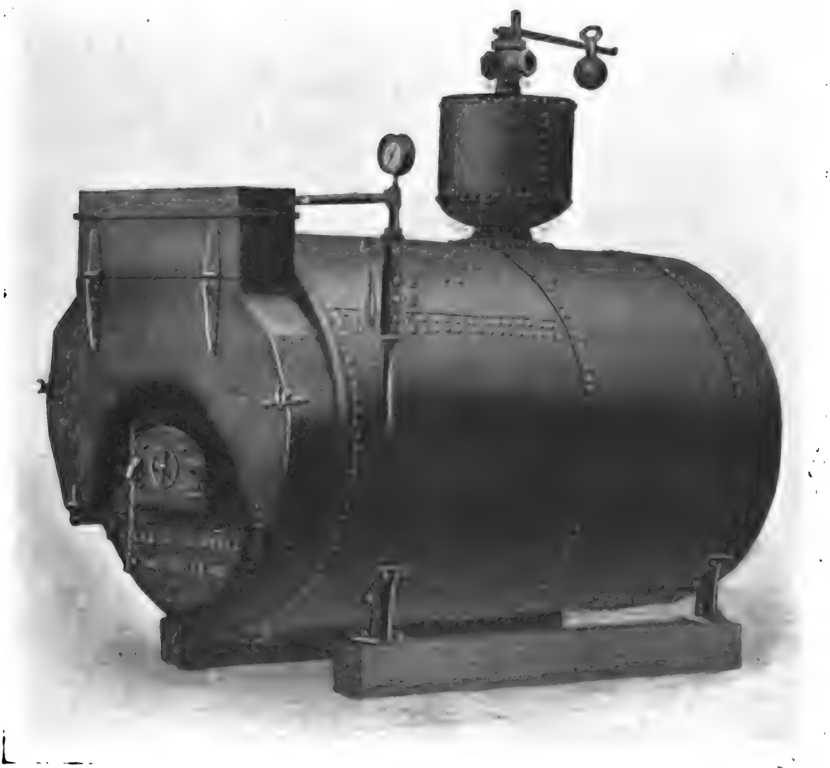


FIG. 76.—INTERNAL FIRED BOILER.

cylinders (Fig. 73) as listed in Table VIII, should be used, and boilers that will carry a pressure of from 150 to 175 pounds steam pressure. They will save about one-third the coal consumed by the piston-pumps. Such pumps will give an initial water pressure of 200 pounds per square inch, and will cut the harder material to better advantage than the ordinary duplex pump. They will require much higher quality of hose and much stronger couplings than for the pumps with lighter pressure. The couplings which are sold on stock hose

are of no value in jetting work, as they are only pressed in, and invariably blow out, so it is best to get special couplings with long

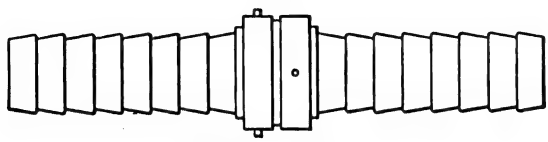


FIG. 77.

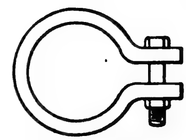


FIG. 78.

nipples to slip into the hose (Fig. 77) that will allow the use of two outside bands on each end (Fig. 78), for the ordinary duplex pump,

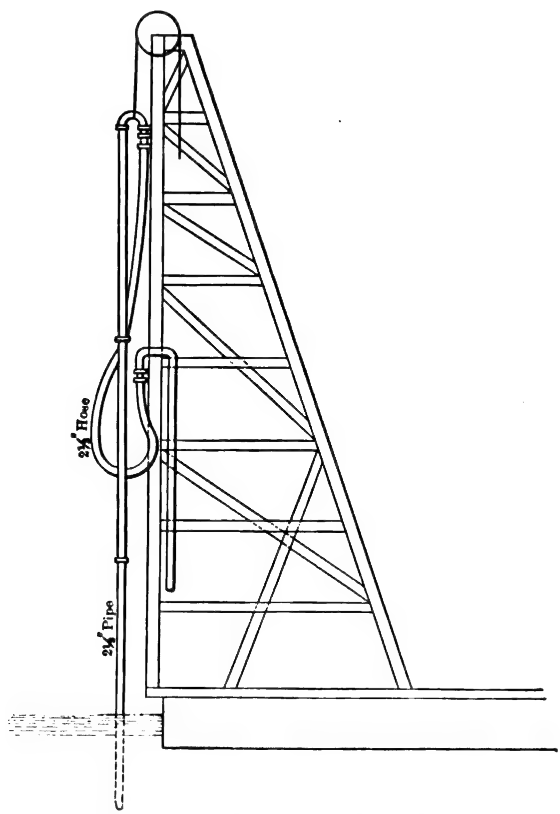


FIG. 79.—JETTING PIPE DETAILS.

and three bands for the higher pressure pumps. These bands are drawn up with bolts as shown and will not cause trouble by blowing

out during the work. The couplings must be provided with pipe-threads. Unless the hose is of the wrapped type, it must be wrapped with burlap or gunny sack wherever it touches a timber or rubs on anything, as the vibrations will soon cut a hole.

The hose should be attached to the jet-pipe by a U and short nipple (Fig. 79) to avoid breaking the hose when under pressure. Where the jet is connected to the piping on a floating driver, large pipe the full size of the pump discharge should be carried up the leads one-third or one-half the height and the hose connected to nipples pointing downward (Fig. 79), so that the hose hangs in one big loop

TABLE V.—PISTON PUMP FOR 150 POUNDS WATER PRESSURE.

Diameter of Steam Cylinders.	Diameter of Water Pistons.	Length of Stroke.	Gallons per Revolution.	Max. Revolutions per Minute.	Max. Gallons per Minute.	Sizes of Pipes for Short Lengths to be Increased as Length Increases. Steam Pipe.	Exhaust Pipe.	Suction Pipe.	Delivery Pipe.	Approximate. Space Occupied. Feet and Inches. Length.		Width.		Size Boilers. Required. for very Heavy. Work. H.P.
2	1¼	2¾	.056	80	4.4	⅜	½	1	¾	1	9¼	0	7	4
3	2	3	.155	80	12.4	⅜	½	1¼	1	2	0	0	9	5
3½	2¼	4	.275	75	23	⅜	¾	1½	1¼	1	9½	0	9	6
4½	2¾	4	.39	75	29	½	¾	2	1½	2	9	1	1	10
5¼	3½	5	.81	70	56	¾	1¼	2½	1½	3	5	1	4	15
6	4	6	1.25	65	81	1	1¼	3	2	3	7	1	5	20
7½	5	6	1.96	65	127	1½	2	4	3	3	9	1	10	35
7½	4½	10	2.60	54	141	1½	2	4	3	4	11	1	10	35
9	5¼	10	3.55	54	192	2	2½	4	3	5	0	1	11	50
10	6	10	4.7	54	254	2	2½	5	4	5	1	2	2	65
12	6	10	4.7	54	254	2½	3	6	5	6	2	3	1	65
12	7	10	6.4	54	346	2½	3	6	6	6	2	3	0	90
14	7	10	6.4	54	346	2½	3	6	6	6	2	3	1	90
12	7½	10	7.39	54	399	2½	3	6	6	6	2	3	0	100
14	7½	10	7.39	54	399	2½	3	6	6	6	2	3	1	100
16	8	10	8.44	54	455	2½	3	6	6	6	7	3	10	105
12	8½	10	9.56	54	516	2½	3	6	6	6	7	3	0	130
14	8½	10	9.56	54	516	2½	3	6	6	6	7	3	1	130
16	8½	10	9.56	54	516	2½	3	6	6	6	7	3	10	130
14	9¼	10	11.37	54	614	2½	3	8	7	6	7	3	1	150
16	9¼	10	11.37	54	614	2½	3	8	7	6	8	3	10	150

An additional charge is made for Tobin-bronze piston rods, brass water pistons, bed plates or for any extras.

The 8½-inch and the 9¼-inch water ends can be fitted with 18½-inch or 20-inch steam cylinders, if desired.

and less length will be required for the run up and down the pile. The line for raising the jets is connected to the U at the top of the jet, run over a sheave on the outside of the head block and down to the nigger head on the engine.

Where deep jetting is to be done as mentioned in the foundations of the Eleventh Street bridge at Tacoma, Volume II, these precautions are doubly important, to save every pound of pressure that might be lost by an unduly long jet line from the pump to the nozzle. The loss of pressure per hundred feet of hose and pipe may be found from Table XI, and can be greatly reduced by using the largest sized pipe and hose up to the jet that can be easily handled. The importance of avoiding loss of pressure may be realized by figuring out the back pressure from the depth to jet the piles, which in the 160 feet below high water in the above case,

TABLE VI.—PACKED PLUNGER PUMP FOR 200 POUNDS PRESSURE.

Diameter of Steam Cylinders, Inches.	Diameter of Water Plungers, Inches.	Length of Stroke, Inches.	Gallons per Revolution.	Revolutions per Minute.	Gallons per Minute.	Sizes of Pipes for Short Lengths to be Increased as Length Increases.				Approximate Space Occupied Feet and Inches.				Size Boilers Required for very Heavy Work. H.P.
						Steam Pipe, In.	Exhaust Pipe, In.	Suction Pipe, In.	Delivery Pipe, In.	Length.		Width.		
10	6	10	4.64	45	220	2	2½	5	4	7	7	2	6	85
12	7	10	6.4	45	300	2½	3	6	5	8		3		115
14	7	10	6.4	45	300	2½	3	6	5	8		3	1	115
14	8½	10	9.56	45	442	2½	3	6	5	8	4	3	2	165
16	8½	10	9.56	45	442	2½	3	6	5	8	4	3	10	165
18½	8½	10	9.56	45	442	3	3½	6	5	8	5	4		165
14	10¼	10	13.95	45	645	2½	3	8	7	8	6	3	6	240
16	10¼	10	13.95	45	645	2½	3	8	7	8	7	3	8½	240
18½	10¼	10	13.95	45	645	3	3½	8	7	8	8	4		240
16	12	10	19.16	45	880	2½	3	10	8	8	7	3	10	330
18½	12	10	19.16	45	880	3	3½	10	8	8	8	4		330
14	8½	15	14.14	40	590	2½	3	6	5	9	5	3	7	225
17	8½	15	14.14	40	590	2½	3½	6	5	9	6	3	11	225
17	10¼	15	20.83	40	855	2½	3½	8	7	10	1	3	11	325
20	10¼	15	20.83	40	855	4	5	8	7	10	2	4	2	325
17	12	15	28.78	40	1175	2½	3½	10	8	10	2	3	11	450
20	12	15	28.78	40	1175	4	5	10	8	10	4	4	2	450

An additional charge is made when the pumps are fitted with Tobin-bronze piston rods and brass plungers with brass-bushed plunger stuffing boxes.

amounted to 69 pounds per square inch, and with 300 feet of pipe and hose the resultant pressure from a pump giving 200 pounds pressure at the pump, would only be from 130 to 140 pounds per square inch at the nozzle, and with a poor arrangement of piping, and hose in bad condition, the pressure at the nozzle might run much below 100 pounds. Some engineers, losing sight of the fact that only a fixed amount of water will go through the jets at a fixed pump pressure, will order larger pumps put on when the jets are not effective, but what is needed in such a case is almost always a greater pump pressure. Where the material is loose gravel or an open boulder bed,

TABLE VII.—COMPOUND PACKED PLUNGER PUMP FOR 200 POUNDS PRESSURE.

Diameter of Steam Cylinders, Inches.	Diameter of Water Plungers, Inches.	Length of Stroke, Inches.	Gallons per Revolution.	Revolutions per Minute.	Gallons per Minute.	Sizes of Pipes for Short Lengths to be Increased as Length Increases.				Approximate Space Occupied Feet and Inches.		Size Boilers Required for very Heavy Work. H.P.
						Steam Pipe, In.	Exhaust Pipe, In.	Suction Pipe, In.	Delivery Pipe, In.	Length.	Width.	
8 & 12	7	10	6.68	45	300	2	3	6	5	9 9	3 1½	90
9 & 14	7	10	6.68	45	300	2	3	6	5	9 10½	3 1½	90
10 & 16	7	10	6.68	45	300	2	3	6	5	10	3 9½	90
12 & 18½	7	10	6.68	45	300	2½	3½	6	5	10	4	90
9 & 14	8½	10	9.8	45	442	2	3	6	5	10 1½	3 1½	135
10 & 16	8½	10	9.8	45	442	2	3	6	5	10 3	3 9½	135
12 & 18½	8½	10	9.8	45	442	2½	3½	6	5	10 3	4	135
14 & 20	8½	10	9.8	45	442	2½	5	6	5	10 1½	4 1⅜	135
10 & 16	10¼	10	14.3	45	645	2	3	8	7	10 6½	3 9½	195
12 & 18½	10¼	10	14.3	45	645	2½	3½	8	7	10 6½	4	195
14 & 20	10¼	10	14.3	45	645	2½	5	8	7	10 5	4 1⅜	195
10 & 16	12	10	19.6	45	880	2	3	12	10	10 6	3 9½	270
12 & 18½	12	10	19.6	45	880	2½	3½	12	10	10 6	4	270
14 & 20	12	10	19.6	45	880	2½	5	12	10	10 4½	4 1⅜	270
9 & 14	8½	15	14.7	40	590	2	3	6	5	10 10	3 6¾	180
12 & 17	8½	15	14.7	40	590	2½	3½	6	5	11 9	4	180
14 & 20	8½	15	14.7	40	590	2½	5	6	5	11 9½	4 1¼	180
12 & 17	10¼	15	21.4	40	855	2½	3½	8	7	12	4	255
14 & 20	10¼	15	21.4	40	855	2½	5	8	7	12 1	4 1⅜	255
12 & 17	12	15	29.4	40	1175	2½	3½	12	10	12	4	360
14 & 20	12	15	29.4	40	1175	2½	5	12	10	12 1	4 1⅜	360

An additional charge is made when pumps are fitted with Tobin-bronze piston rods and brass plungers with brass-bushed plunger stuffing boxes.

TABLE VIII.—LOCOMOTIVE BOILERS WITH WATER BOTTOM.

Nominal Rated Horse Power.	Shell.		Fire Box.			Dome.		Size of Steam Outlet. In.	Tubes.			Total Heating Surface Sq. Ft.	Stack.		Shipping Weight. Pounds Approximate.		
	Diam. In.	Length over all Ft. In.	Length. In.	Width. In.	Height. In.	Diam. In.	Height. In.		No.	Diam. In.	Length. Ft. In.		Diam. In.	Length. Ft.	Boiler only.	Fixtures only.	Boiler and Fixtures.
10	28	9 5	30	23	36	15	16	1½	25	2½	5 6	127	10	15	2754	446	3200
15	32	11 0	42	27	40	15	16	2	34	2½	6 0	188	12	15	3829	671	4500
20	34	12 8	48	29	45	15	16	2	40	2½	7 0	251	12	20	4902	798	5700
25	36	14 0	52	32	48	18	20	2½	45	2½	8 0	315	14	25	5832	968	6800
30	40	15 2	54	36	48	20	22	2½	49	2½	9 0	376	18	35	6636	1364	8000
40	42	16 4	54	37	60	20	22	3	62	2½	10 0	510	20	40	8445	1555	10000

Each boiler, on skids, is furnished with dome, grate bars, safety valve, steam gage and siphon, glass water gage and gage cocks, check valve, stop valve blow-off cock, whistle, smock stack, and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

TABLE IX.—LOCOMOTIVE BOILERS WITH OPEN BOTTOM.

Nominal Rated Horse Power.	Shell.		Fire Box.			Dome.		Size of Steam Outlet. In.	Tubes.			Total Heating Surface. Sq. Ft.	Stack.		Shipping Weight. Pounds Approximate.		
	Diam. In.	Length. Over all Ft. In.	Length. In.	Width. In.	Height. In.	Diam. In.	Height. In.		No.	Diam. In.	Length. Ft.		Diam. In.	Length. Ft.	Boiler only.	Fixtures. only	Boiler and Fixtures.
50	50	17 6	60	44	46	24	24	3	60	3	11	600	22	40	9500	2000	11500
60	52	18 6	60	46	48	28	26	3½	68	3	12	725	24	40	10922	2078	13000
70	54	19 6	60	48	50	28	26	3½	76	3	13	870	26	40	12670	2330	15000

Each boiler, on skids, is furnished with dome, grate bars, safety valve, steam gage and siphon, glass water gage, and gage cocks, check valve, stop valve, blow-off cock, whistle, smoke stack and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

TABLE X.—INTERNAL FIRED BOILERS WITH INDEPENDENT DOME.

Nominal Rated Horse Power.	Shell.			Furnace.		Mean Thickness.			Dome.		Size of Steam Outlet. In.	Return Tubes.			Total Heating Surface. Sq. Ft.	Stack.		Shipping Weight. Pounds Approximate.		
	Diam. In.	Length over all Ft.	In.	Diam. In.	Length. In.	Shell. In.	Heads. In.	Flue. In.	Diam. In.	Height. In.		No.	Diam. In.	Length. Ft.		Diam. In.	Length Ft.	Boiler only.	Fixtures only	Boiler and Fixtures.
10	40	9	7	20	30	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{3}{8}$	15	16	$1\frac{1}{2}$	16	3	8	138	12	25	4529	471	5000
15	46	9	7	24	40	$\frac{1}{4}$	$\frac{7}{16}$	$\frac{3}{8}$	15	16	2	24	3	8	196	14	25	5513	687	6200
20	52	9	7	26	46	$\frac{9}{32}$	$\frac{7}{16}$	$\frac{13}{32}$	15	16	2	30	3	8	240	16	25	6621	779	7400
25	56	9	7	28	50	$\frac{9}{32}$	$\frac{1}{2}$	$\frac{13}{32}$	18	20	$2\frac{1}{2}$	38	3	8	300	18	30	7555	1045	8600
30	60	9	7	30	50	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{7}{16}$	20	22	$2\frac{1}{2}$	47	3	8	362	20	30	8818	1182	10000
40	66	9	8	34	56	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	24	24	3	68	4	8	496	24	30	10513	1387	12100

Each boiler, on skids, is furnished with dome, grate bars, safety valve, steam gage and siphon, glass water gage, and gage cocks, check valve, stop valve, blow-off cock, whistle, smoke stack and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

TABLE XI.—FRICTION LOSS IN JET PIPES AND HOSE, AVERAGE SMOOTHNESS

Diameter.	Gallons per Minute Flowing.																						
	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000	1050	1100	1150	1200
	Friction Loss, Pounds per 100 Ft. Length																						
2	14	30	55	85	120																		
2½	5	10	18	28	40	54	72	90	112														
3	2	4	7	12	16	22	30	40	45	54	65	76	88	100	115								
3½	..	2	3	5	8	10	13	16	20	24	30	34	41	45	53	58	65	72	83	88	97	106	115
4	..	1	2	3	4	5	7	8	11	12	15	17	21	23	28	30	33	36	40	44	4	52	56
5	..	..	..	..	1	2	2	3	4	4	5	6	7	8	9	10	11	13	14	15	17	18	20
6	..	..	..	..	..	..	..	1	1	2	2	2	3	3	4	4	5	5	6	6	7	7	8
7	..	..	..	..	..	..	..	..	..	1	1	1	1	1	2	2	2	2	3	3	3	3	4
8	..	..	..	..	..	..	..	..	..	...	...	...	...	...	1	1	1	1	1	2	2	2	2

Values were taken from a diagram, to the nearest pound. Table shows that for large-sized pipe, friction loss may be practically disregarded, within the capacities of pumps up to 8 inches discharge.

the water will dissipate through the bottom and fail to lubricate the sides of the pile and the jetting plant be almost valueless, and the use of larger pumps be of little avail, unless of course the hose, piping and nozzle are made very much larger so as to actually deliver a much larger volume of water to overcome the seepage. The largest pipe that can be reasonably handled for the jets themselves is 3 inches in diameter, and the nozzle could be increased to 1½ inches or 1¾ inches. But for compact material where the water will not be lost, and for depths out of the ordinary, it is pressure that is needed, and the required pressure to accomplish the desired results may be obtained by a careful consideration of the conditions and a common-sense layout of the plant and piping. The required pump pressure may be taken from the fire stream data as given in Table XII, for various sized nozzles.

TABLE XII.—DISCHARGE IN GALLONS PER MINUTE FROM NOZZLES ATTACHED TO 50 FT. OF 2½-IN. SMOOTH-LINED HOSE.

Pressure at Pump.	Size of Smooth Nozzle.							
	1¾	1½	1⅜	1¼	1⅛	1	⅞	¾
10	195	165	145	125	105	85	70	50
20	275	230	205	180	150	125	95	70
30	335	285	250	220	185	150	120	90
40	390	325	290	255	215	175	135	100
50	430	365	325	285	240	195	150	115
60	475	400	355	310	260	215	165	125
70	510	430	385	335	280	230	180	135
80	545	460	410	355	300	245	190	145
90	580	490	435	380	320	260	205	150
100	610	515	460	400	335	275	215	160
110	640	540	480	420	350	290	225	170
120	670	570	505	440	370	300	235	175
130	700	590	530	455	385	315	245	180
140	730	615	550	475	400	330	250	190
150	760	640	570	495	420	340	260	195
160	790	660	590	510	430	350	270	200
170	820	680	610	525	445	360	280	205
180	850	700	630	545	460	370	285	210
190	880	720	650	560	475	380	290	215
200	910	740	670	580	490	395	300	220

Values read from diagram to nearest five pounds.

Unless the borings made in the original soundings for the foundations prove to be reliable, and from which it is presumed the pumping plant will be designed, much experimenting will have to be done to

arrive at the best arrangements that can be made, and should changes be found necessary the bottom should be sounded thoroughly at the site of each pier with the jets, which will best develop the actual conditions to be met with. In some instances good results have been obtained in a somewhat porous bottom, by using a large pump and perforating the sides of the jet pipe every few feet with $\frac{1}{8}$-inch holes. This can only be done at the bottom end, however, as the water must be shut off before these holes come above the surface of the water to avoid a deluge over the men and plant, and shutting off the water is dangerous, as the jets are then almost sure to freeze or stick in the bottom, so they cannot be pulled out, and are thus lost for future use.

TABLE XIII.—HEAD IN FEET WITH EQUIVALENT IN POUNDS PRESSURE PER SQUARE INCH.

Head	Pressure	Head	Pressure	Head	Pressure	Head	Pressure	Head	Pressure
1	.434	29	12.58	57	24.73	85	36.89	165	71.61
2	.868	30	13.02	58	25.17	86	37.32	170	73.78
3	1.30	31	13.45	59	25.60	87	37.75	175	76.90
4	1.73	32	13.88	60	26.04	88	38.29	180	78.12
5	2.17	33	14.32	61	26.47	89	38.62	185	80.29
6	2.50	34	14.75	62	26.90	90	39.06	190	82.46
7	3.03	35	15.19	63	27.34	91	39.49	195	84.63
8	3.47	36	15.62	64	27.76	92	39.92	200	86.80
9	3.90	37	16.05	65	28.21	93	40.36	210	91.14
10	4.34	38	16.49	66	28.64	94	40.79	220	95.48
11	4.77	39	16.92	67	29.07	95	41.23	230	99.85
12	5.20	40	17.36	68	29.51	96	41.66	240	104.16
13	5.65	41	17.79	69	29.94	97	42.09	250	108.50
14	6.07	42	18.22	70	30.38	98	42.53	260	112.84
15	6.51	43	18.66	71	30.81	99	42.96	270	117.66
16	6.94	44	19.09	72	31.24	100	43.40	280	121.52
17	7.37	45	19.53	73	31.68	105	45.57	290	125.86
18	7.81	46	19.94	74	32.11	110	47.74	300	130.50
19	8.24	47	20.39	75	32.55	115	50.91	350	152.20
20	8.68	48	20.83	76	32.98	120	52.08	400	173.60
21	9.11	49	21.26	77	33.41	125	54.25	450	195.30
22	9.54	50	21.70	78	33.85	130	56.45	500	217.00
23	9.88	51	22.17	79	34.28	135	58.62	600	260.40
24	10.41	52	22.56	80	34.72	140	60.76	700	303.80
25	10.85	53	22.80	81	35.15	145	63.93	800	347.20
26	11.06	54	23.43	82	35.58	150	65.10	900	390.60
27	11.71	55	23.87	83	36.02	155	67.27	1000	434.00
28	12.15	56	24.30	84	36.45	160	69.44	1500	651.00

The use of jets either from a pile-driver scow or a pump scow (Fig. 70) is the most satisfactory, as it employs a low lift for the

pumps, gives a short delivery line, which reduces the losses at both ends, and results in a much greater pressure at the nozzle than is the case with a pumping plant located in one position on shore or on falsework. The jets should always be run down into the bottom, as has been stated, and then the pile dropped in, the jets kept going up and down the pile as it is tapped with the hammer and driven home.

The same pump scow should be rigged to handle the sand pumps or hydraulic elevators on a piece of work similar to that described in Volume II, an extra pump being provided not less than 10×6×10 to supply a jet to loosen up the material for the pump or elevator. Whether the sand pump is used for excavating the crib or not, it is useful in cleaning out for concreting, and especially necessary where piles have been driven in the crib, to remove the material swelled up in driving and jetting the piles. While it is usual to dredge out a crib somewhat deeper than required to allow for this, the jetting will often fill up a crib from 4 feet to 10 feet with soft stuff which must be removed to have the bottom in proper condition to receive the concrete.

Where cribs are to be sunk to great depths, it is advisable to have a pumping plant to supply water to jets for jetting around the crib to reduce the friction. This can either be done by separate jets or by jets built into the crib or caisson, and this might require a much larger equipment of boilers and pumps than would be needed for the jetting of piles.

The depth to which it is necessary to jet piling in a bridge foundation is determined from one of three conditions, first, to get below scour; second, to reach a hard substratum which is to carry the load; and third, to a sufficient depth in a soft bottom so that the frictional resistance of the pile will carry the required load.

The depth to go in the first case must be arrived at from a careful study of the location as to the depth scour will reach, bearing in mind the changed flow that will result in placing piers in the river. If there is any question about this, the piles must be given a penetration of a sufficient amount to make the piers safe beyond question, and riprap must be placed around the pier, so as to prevent scour starting.

The probable result of such study will doubtless indicate that the crib or caisson must be carried deeper, as scour below the cutting edge would lay the piles bare and make the safety of the pier doubtful.

The second case, where piles are required to be jetted down to

reach a hard substratum, would cause the piles to act as timber columns, and consequently would be very much limited as to the safe or economical depth. Probably not over 40 to 50 feet would be considered safe for the ordinary-sized piling, for the ordinary loads.

If piles are considered as columns, the figures would sometimes indicate that they would carry a greater load than indicated by the friction, or the usual formula $\frac{2wh}{s+1}$, so they must be used with very careful judgment, especially as the pile in reaching hard bottom may rest on the point only. See Appendix XI.

Where the engineer can be sure that the material through which the piles have been driven to the hard bottom is firm enough to give sidewise support to the pile, they can be figured as short columns (Table III, Chapter IV) to carry a load to 1000 pounds per square inch as given by Rankine, or the lesser value of 786 pounds as given by Peronnet for ordinary, good timber. The values for various classes of timber as shown by recent tests are given in the tables of timber in Chapter XV.

The depth to go in the third case where the friction on the piles must carry the load, is to be determined by applying the formula for driving with a drop-hammer or a steam hammer as the case may be, until the pile is driving hard enough to carry the load. Where the piles are being jetted, it is the usual custom to stop jetting at about the required depth, and hammer the pile for some little time to see what the penetration per blow is after getting below the material disturbed by the jet, in order to apply the formula.

In many cases where piles have been jetted and allowed to set for some hours until the material has frozen around them, they cannot afterwards be moved with the hammer, or at least very little penetration will result, which fact goes to show that in many cases piles are driven to ridiculous depths simply because the jets will assist them there. In no case should they be jetted deeper than necessary to carry the load by the friction on the surface, as set forth in Chapter IV, that is, 500 pounds per square foot of surface in firm sand or clay, or as low as 120 pounds per square foot in silt. Careful actual tests in any case where a large number of piles are to be driven might indicate that these values could be doubled or trebled, with perfect safety. See Appendix XI.

CHAPTER VI

CONSTRUCTION WITH SHEET-PILES *

WATER pressure against the sides of a sheet-pile coffer-dam is seldom provided for in an accurate manner, the thickness of the piling being usually decided upon from past experience, as are also the size and spacing of the guide-piles and wales.

These are points where guess-work should be eliminated, as otherwise good coffer-dams are often seen where the pressure has so bulged the plank as to cause leakage. While this may perhaps be corrected by additional bracing, simple calculations may easily be made to determine the size beforehand.

The pressure against a coffer-dam may act as at (*a*), Fig. 80, the sheet-piling being in the condition of a beam fixed at one end and loaded with a gradually increasing weight, as shown by the dotted lines, due to the pressure of water or puddle at 62.4 pounds per cubic foot. Then the load on a width w of the wall is $124.8wd^2$ and the moment of the pressure is $83.2wd^3$. Taking the allowable unit stress on wet timber at 1000 pounds per square inch, the thickness t of the sheet-piling may be obtained from the formula

$$t=\sqrt{.496d^3},$$

in which d is to be taken in feet, and the resulting value of t will be the thickness in inches of the sheet-piling.

This formula has been expressed in a graphic manner in diagram (*d*), Fig. 80, from which, knowing the depth of water $2d$. the thickness of piling may be read directly without calculation.

The addition of a strut, as at (*b*), Fig. 80, places the sheet-piling in the condition of a beam supported at the upper end and fixed at

* The assumption that the pressure of puddle will be the same as water pressure is made advisedly. It is true that *very* wet clay, approaching a fluid condition, will exert a much greater pressure, but it would then be useless as puddle. Dry clay would exert a pressure of less than half that due to water, so it has been assumed that *wet* clay or puddle would exert the same force as water. Should it exceed it for a short time no damage would be done, owing to the low unit stress adopted.

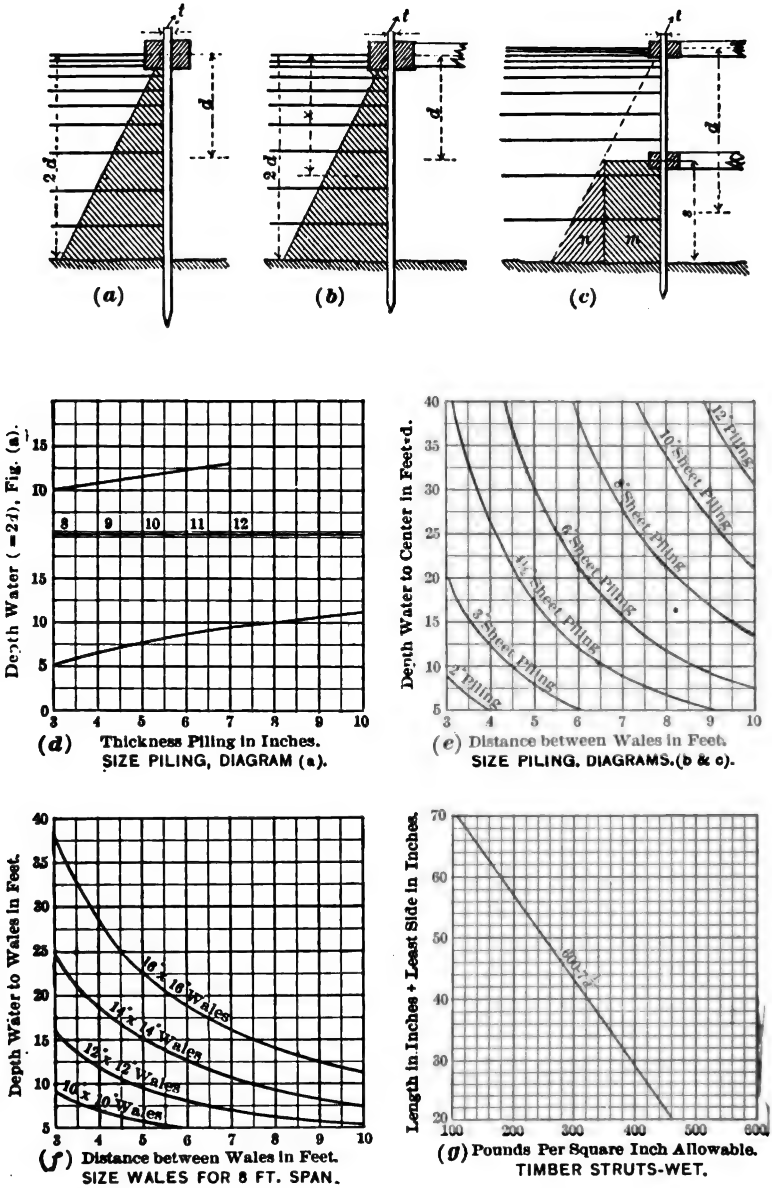


Fig. 80.—Arrangement and Diagrams of Sizes for Sheet-pile Coffer-dams.

the lower end, but for practical reasons it is best to consider it as merely supported at both ends. The load will be the same as in the former case, $124.8wd^2$, but the maximum moment will occur at a point x, which is a distance from the top equal to 1.16 times d, and has a value of $32wd^3$. The thickness t may be found from the formula

$$t=\sqrt{.19\text{[illegible]}^3}.$$

When the section of the plank to be calculated is located as "s" in (*c*) of Fig. 80, it is in the condition of a beam fixed at both ends and loaded with a uniform load m and a triangular load n. The exact analysis of this is too lengthy to be taken up here, and reference may be made to page 195 of Wood's "Resistance of Materials."

For practical purposes we may consider the load as all uniform and due to the head acting at the middle of the span. This will give a load of $62.4wds$ on the span s for a width w, and a moment of $7.8ds^2$, which gives a formula for practical use, for a unit stress of 1000 pounds per square inch of

$$t=\sqrt{.047ds^2}.$$

This is closely represented graphically in diagram (*e*) of Fig. 80 which may also be used for case (*b*) by taking the depth of water to the middle of the span. For example, when the depth of water to the middle of the span is 15 feet, find this in the vertical column to the left, and if 6-inch sheet-piles are to be used, follow the horizontal through 15 feet until it intersects the 6-inch curve and vertically beneath will be found the maximum spacing of wales, 7 feet 3 inches.

The size and spacing of wales may be taken from a similar diagram (*f*) of Fig. 80, which assumes the guide-piles to be 8 feet apart. The spacing of struts or braces will vary so much that the load must be calculated, and when this and the length are known the size may be calculated from diagram (*g*) of Fig. 80, which is for wet timber.

From the formula

$$p=600-7(l\div d),$$

in which p is the allowable stress in pounds per square inch, l is the unsupported length in inches, and d the least side of the stick in inches.

Where two rows of sheet-piling are to be driven to form a puddle-chamber, if they are to be efficiently braced from the inside of the coffer-dam, it will be sufficient to have a thickness of puddle of from

2 to 4 feet to exclude the water, depending on the quality of the puddle. Where there is to be no internal bracing, but two rows of sheet-piling braced together and filled with puddle are to resist overturning, the common rule is to make the width of the puddle-chamber equal to the height above ground, up to 10 feet. When the height exceeds 10 feet, add one-third to the excess height to 10 feet for the width.

When the puddle-chamber becomes very wide it is often divided into several compartments, as was shown in Fig. 5, and stepped in a similar manner. When the bottom is rock overlaid with a thin deposit of clay or gravel, the sheet-piles may be driven around an open crib-work for support, as was done at Harper's Ferry, on the B. & O. R. R.

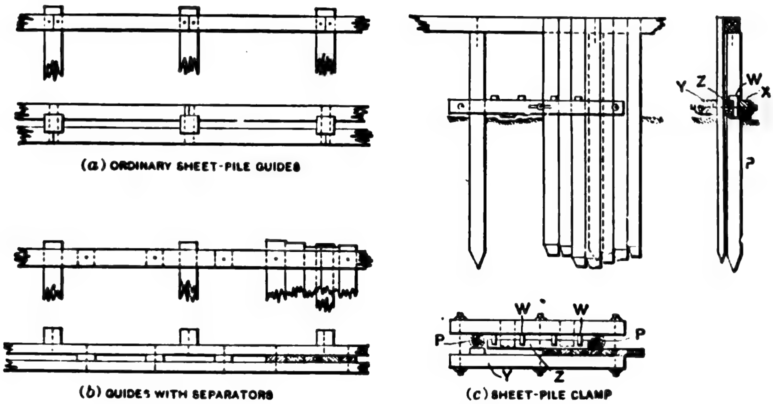


FIG. 81.—SHEET-PILE GUIDES AND CLAMPS.

Where the guide-piles are to be used, the waling-pieces are framed in, as was specified on the Hutcheson Bridge, as shown at (*a*), Fig. 81, where the guide-piles are of sawed timber. The wales are spaced slightly farther apart than the thickness of the sheet-piles, to allow clearance in driving, the space between the guide-piles being filled out with a key pile to fill the panel tightly. This method is but little used with tight piling, that shown at (*b*), Fig. 81, allowing the piling to be driven continuously, by removing the spacing blocks as they are reached, and substituting bolts through the sheet-piles, firmly connecting the piles and wales together.

A very satisfactory method is described in the *Engineering News* of May 12, 1892, which was used by A. F. Walker. Having occasion to do a large amount of work it was desirable not to go to the expense

of squared guide-piles. Round guide-piles (*P*) were first driven 7 feet apart, and cut off to a level. Caps were then drift-bolted to the tops, allowing them to project slightly beyond the face of the round piles, thus forming a permanent support for the top of the sheet-piles. Near the ground line was placed the clamp, consisting of two sticks (*X*) and (*Y*), connected by three bolts and drawn together as tight as the intervening piles or pile and gage-block (*G*) will permit. The stick (*Z*) is then forced forward by the wedges (*W*) until the space between (*Z*) and (*Y*) is the same as the thickness of the piles. The pieces (*X*), (*Y*), (*Z*) are slotted for the middle bolt, and this permits of some adjustment. When one of the piles partially closes this slot, a notch is cut in it large enough to receive the bolt, and the bolt is then slipped up to it and tightened. This allows of the next pile being driven as close as the others. When one panel has been completed the nuts are removed and the clamps moved forward to the next one, a notch being cut in the end pile to receive the end bolt of the clamp. The piles are sharpened flatwise with a little more slope on the side facing the guide-piles, giving them a tendency to drive away from the guide-pile at the foot and bear against the cap at the top. A slight bevel is also given to the edge to make the foot crowd the adjoining pile. During the first half of the driving, the joint is held a little open at the top, but during the latter half, pressure is brought to crowd it toward its neighbor, and the joint will close as tightly as possible.

The use of single pieces of timber as wales, against which the sheet-piling is driven, is illustrated in the use of method (*b*) of Fig. 52, by Benj. Douglas, bridge engineer of the Michigan Central Railway. The coffer-dam (Fig. 82) was built without guide-piles, the wales being 12×12-inch timber bolted against the outside of the sheet-piling, by the brace rods 1 inch in diameter. The wales are held in place vertically by bracing of 2×12-inch pine plank, which are spiked on as verticals and diagonals to form a truss and also to stiffen the framework in general.

The sheet-piling is 6×12, and after being driven into the hard gravel bottom, the cracks were lapped by 1-inch boards. The bottom was uneven and accounts for the difference in height, the excavation at the high end being dumped outside at the low end, to assist in making the dam tight. The puddle-chamber was 2 feet 8 inches wide and was filled with clayey gravel. The plan also shows the grillage in place for receiving the foundation courses of the stone-work. This is formed by 12×12 timber crossed, and drift-bolted together with 1-inch round and 18-inch long drift-bolts.

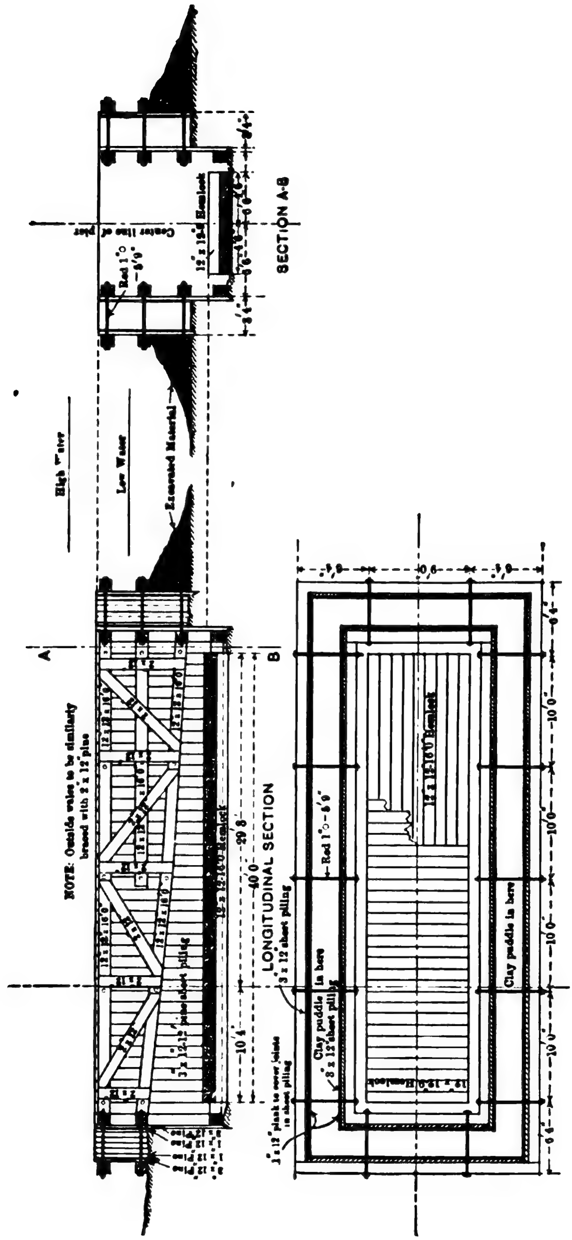


FIG. 82.—COFFER-DAM FOR ANN ARBOR BRIDGE. MICHIGAN CENTRAL RAILWAY.

The account of the Arthur Kill Bridge foundation in Vol. 27 of the "Transactions of the American Society of Civil Engineers," by A. P. Boller, consulting engineer, covers a very interesting experience with sheet-piling on pier No. 5: "This pier is near the edge of the marsh forming the Staten Island shore, which is barely flooded at extreme high tides. Borings indicated about 30 feet from the surface to hard bottom, consisting of mud, mud and clay, clay and shale to the bottom of shaley clay, in which the pier was to be founded. Experience on other work of a similar character indicated that the founding of this pier would be accomplished with little difficulty. The area of the foundations was inclosed with a tongued-and-grooved sheet-pile dam of 4-inch yellow pine plank. But it was found impossible to hold the plank at a depth of 15 feet, the mud and clay becoming puddled with water, and despite all efforts at bracing, the plank shoved inward to such an extent as to spoil the whole dam before we were half way down. A second dam was therefore driven around the first one, but this time with 10×12-inch tongued-and-grooved timbers, in one length to reach the extreme bottom. These timbers were grooved by slitting the grooves out at the mill with a circular saw, and chiseling the blank so formed free. The tongue was an independent spline, $2\frac{1}{2}\times 4$ inches, of dry wood and nailed in one groove. The timbers were shaped at the feet to drive close. This dam was hard driving but was finally accomplished, when digging was resumed and the old dam removed piecemeal as we could get in the braces. The bottom was reached within a perfect dam, with only one bad leak in the northwest corner, due to the shattering of a small piece of one tongue during the driving. As it was impossible to stop this leak from the inside, and the outside was inaccessible, to prevent washing the concrete, the leak was led off in a box at the side of the dam to the sump-well, and the footing course of concrete, filling the whole area of the dam about seven feet deep, was gotten in place."

This example emphasizes in a very decided manner many of the statements that have been made heretofore. While no doubt the removal of the old dam was attended with much expense, its inclosure entirely within the new sheet-piling rendered the prosecution of the work comparatively certain.

An example of the driving of sheet-piling on a slant, to prevent crowding in at the bottom, is shown in Fig. 83, which is a cross-section of a sewer coffer-dam used on the Metropolitan Sewerage Systems of Massachusetts by Howard A. Carson, chief engineer, and described in the *Engineering News* of Feb. 8, 1894.

The outlet into the ocean at Deer Island begins at a point about 60 feet inside the high-water line, and about 1850 lineal feet is from 5 to 10 feet below high water. This necessitated the coffer-dam, which was constructed with bents every 6 feet and with 2-inch plank inside the high water-line, but for the remaining distance of 4-inch matched plank. The excavation was done by means of buckets, traveling-derricks, and dump-cars, the latter being emptied at the sides and ends of the trench. The leakage from the ocean was kept out by using centrifugal pumps, which pumped a maximum of 46,000 gallons per hour. The concrete, which has large boulders embedded in its surface the size of paving-stones, was carried up to the level of the ocean bottom.

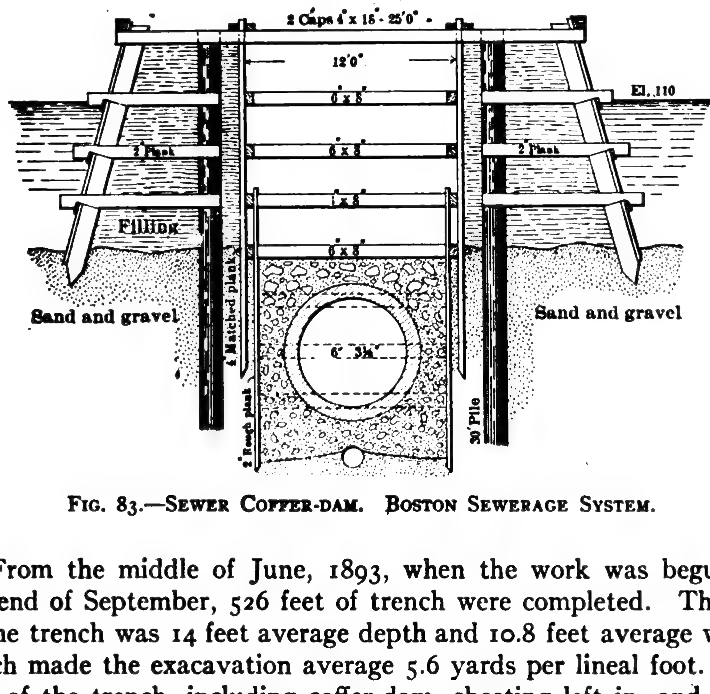


FIG. 83.—SEWER COFFER-DAM. BOSTON SEWERAGE SYSTEM.

From the middle of June, 1893, when the work was begun, to the end of September, 526 feet of trench were completed. The size of the trench was 14 feet average depth and 10.8 feet average width, which made the exacavation average 5.6 yards per lineal foot. The cost of the trench, including coffer-dam, sheeting left in, and back-filling, was $44 per lineal foot.

Casual mention has been made in several places of the use of Wakefield sheet-piling which was illustrated at *h* and *h'* of Fig. 52 and which is further shown in Fig. 84. View No. 1 is of a corner which is formed as in the plan No. 2, a tongue being bolted on the side of a pile, when the corner is reached as in No. 3. Any angle is turned by a similar method, which is shown by No. 4, or the piles

may be driven to form a curve. The essential features of the system are the triple lap or long tongue and groove which excludes the

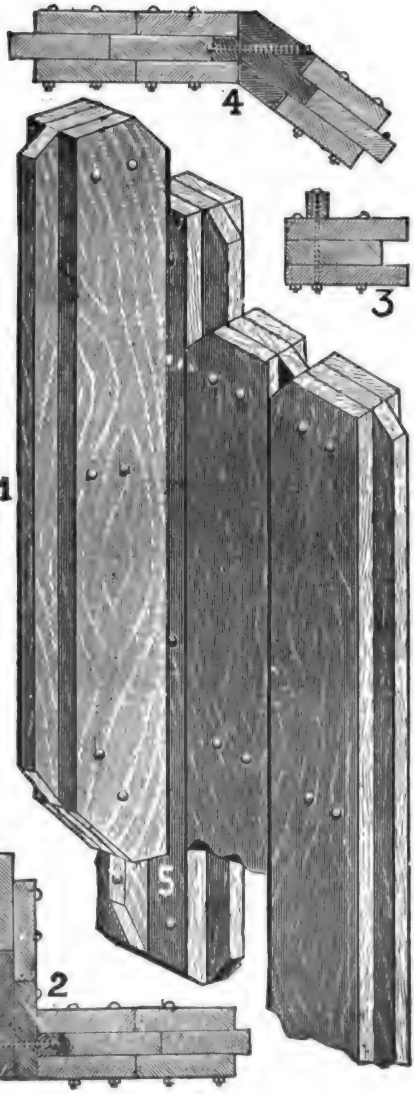


FIG. 84.—WAKEFIELD SHEET-PILING.

water, and the use of ordinary plank, which can be easily obtained. The center plank should be sized to a uniform thickness, to insure the tongues fitting the grooves, and to make driving easy, while the

three plank are to be bolted and spiked together to cause them to act as a compound beam and not as separate plank like the system of (*b*), Fig. 52. It is recommended to use a 2½-inch tongue on 1-inch boards and ⅜-inch bolts. For 1½-inch plank a 3-inch tongue, for 2-inch and 2½-inch plank a 3½-inch tongue and ½-inch bolts, while for 3-inch plank a 3½-inch tongue and ⅝-inch bolts are to be used, and the same size bolts for 4-inch plank, but a 4-inch tongue. Two bolts are to be staggered in every 5 to 8 feet of the length of the pile, and spikes used between the bolts on long piles.

The La Grange lock on the Illinois River was inclosed with this piling, under the direction of Major W. L. Marshall, Corps of Engineers. It was intended to back the sheeting with earth, but as both dredges broke down the water-tightness was entirely dependent on the Wakefield piling, and under a 7-feet head no leaks were developed. The piles were made of three plank 3×12 inches by

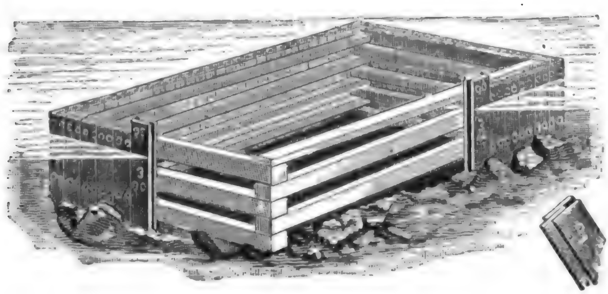


FIG. 85.—TYPE OF MOMENCE AND HARPER'S FERRY COFFER-DAMS.

22 feet long and with a 3-inch tongue; they were driven by three pile-drivers with hammers of from 2800 to 3000 pounds through sand and mud, and in one place a layer of shells. There was no difficulty experienced in driving the piles without special appliances.

The use of 1-inch boards in this form (Fig. 85) is described by H. F. Baldwin, chief engineer of the C. & E. I. Railway: "In constructing our second track over the Kankakee River at Momence, Ill., it was necessary to extend the piers in that river. The bottom is limestone and the surface is very irregular. We tried several days and finally succeeded in constructing a coffer-dam with two parallel walls of sheet-piling. We then tried the Wakefield triple lap-piling, constructed of 1-inch boards sharpened to an edge, 2½ tongue and groove, which were driven with sledges until the piles, which were soft pine, conformed to the uneven surface of the rock. This piling was driven around cribs loaded with stone, and, after the piling

was driven, gravel was put outside the coffer-dam, after which no trouble was experienced in pumping out the water."

The work on the foundations of the new B. & O. R. R. bridge over the Potomac River at Harper's Ferry was similar in many respects to the above, and the system was found to be very satisfactory.

References were made to the use of this piling on the Charlestown Bridge at Boston and the driving of the piles shown in Fig. 53. The work was under the charge of Jno. E. Cheney, consulting engineer,

FIG. 86.—COFFER-DAM ON CHARLESTOWN BRIDGE.

and was successfully carried out. The piling were driven principally as forms for concrete foundations, and but little care was taken to make the dams water-tight. After the concrete was deposited they were used as coffer-dams against a 6- or 7-feet head of water. They were 18 feet 6 inches by 119 feet (Fig. 86) and in some cases were 30 feet below low water or 40 feet below mean high water. The piling was made of 2-inch plank and driven with an ordinary pile-driver. The pumping was done with a 20-inch centrifugal pump, and in some cases a 12-inch Follansbee pump of the propeller type was used.

The construction of the sewerage system at Fort Monroe, Va.,

under Capt. Thos. L. Casey, Corps of Engineers, is described in the report of the Chief of Engineers of 1896. The work was done on the general plans of Rudolph Hering, consulting sanitary engineer. One of the special difficulties encountered "was the building of a sewage tank 50 feet in diameter, with walls of brick 2 feet in thickness, exteriorly diminishing to 3 feet at the center, the inferior reference of which was 20 feet below low water. As described in the report referred to, this was accomplished very successfully by excavating a large area to the reference of ground-water, some 5 or 6 feet below

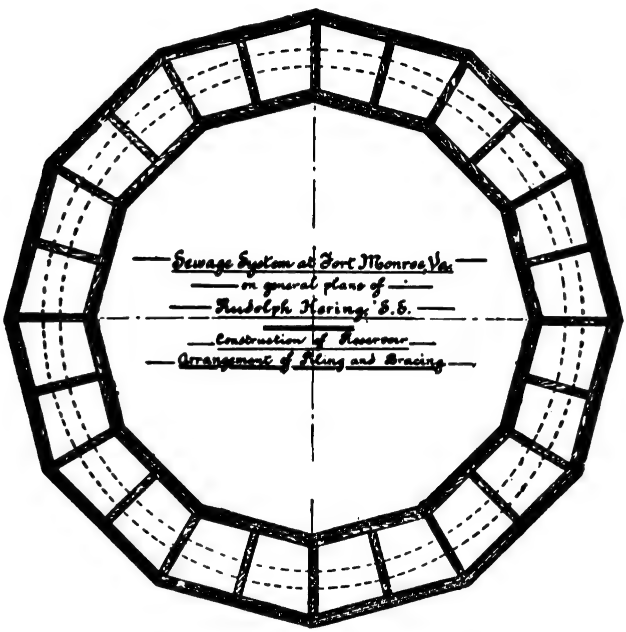


FIG. 87.—RESERVOIR COFFER-DAM. FORT MONROE, VA.

the surface, and then driving by the pile-driver and water-jet combined, two concentric twelve-sided polygons of Wakefield sheet-piling 28 feet in length, 30 and 22 feet from the center, about the circumference of the shallow excavation. (Fig. 87.) The material, consisting of fine water-soaked sand, with a small admixture of clayey matter and fine gravel, was then excavated between the polygons to a reference of 20 feet, transverse shoring braces bearing upon stout stringers being put in at intervals as the work proceeded. The material did not vary much in its general nature, but a number of old piles were taken up, some of which did considerable injury

to the sheet-piling when driven, as shown in the subsequent excavation. The water was controlled by a powerful steam-pump having its point of suction fixed, the water being permitted to flow toward it throughout the circumference. It was noticed that ground-water came through the sheeting very freely at first, but that it constantly ceased to flow to any great extent at a height of a few feet above a point of excavation as this continually descended, owing to the rapid drainage of the strata. The interior core, in fact, became quite dry, so that in excavating after the walls were laid, no water was encountered until the bottom of the external concrete ring had been virtually laid bare. Upon attaining the reference—20 feet, the excavation ceased and hand-mixed concrete was deposited directly upon the bottom, as this was considered to be sufficiently firm, the pump being stopped temporarily in order to prevent a flow. The concrete was rammed firmly against the outer sheeting externally and against plank forms with triangular cross-section resting against the inner sheeting internally, until 6 feet in depth had been put in place. The portion of the ring at the pump suction was filled rapidly with concrete in bags. The 2-feet brick wall was then carried up from the axial line of the concrete ring, the space between the wall and the outer sheeting filled with sand, except about 6 inches at the base of the wall, which was of concrete. The braces were removed as successively attained, the inner prism of dry sand being held securely by the sheeting and the extreme top struts, which were left in place until the inner core was completely excavated. On the completion of the latter work to reference—20 feet, the water which came in freely from without under the concrete ring at several points was conducted in a peripheral trench to the fixed point of pumping. No water came upward and the middle portions of the bottom became perfectly dry. The inner sheeting was cut off at the base of the ring, boards were placed transversely over the peripheral trench, a duck tarpaulin coated with hot asphalt laid down, and concrete rammed in place until the concave bottom with sump channel had been completed, leaving only the pipe, through which the ground-water had been pumped continually, night and day at about 1000 gallons per minute, penetrating the concrete. In order to fill this pipe, it was cut off above the level of permanent ground-water, and after the water within had attained the level of ground-water in the surrounding area and had become perfectly quiescent, neat cement in paper bags was dropped within, being retained at the bottom by the closed valve; the bags were readily broken up by a long pole thrust down the pipe. The latter was then cut off at the level of the bottom

and a coating of cement plaster applied throughout. The resultant leakage through the bottom did not exceed about a gallon a minute and this will be greatly reduced by the infiltration of sand from beneath."

Further illustrations of the use of sheet-pile coffer-dams will be given; then the operations of dredging, pumping, and concreting will be described at some length.

The piers for a bascule bridge over the Lake Washington Canal in Seattle constructed by the author for the Northern Pacific Railway under W. L. Darling, Chief Engineer of the road, were built from plans prepared by H. E. Stevens, Bridge Engineer of the line.

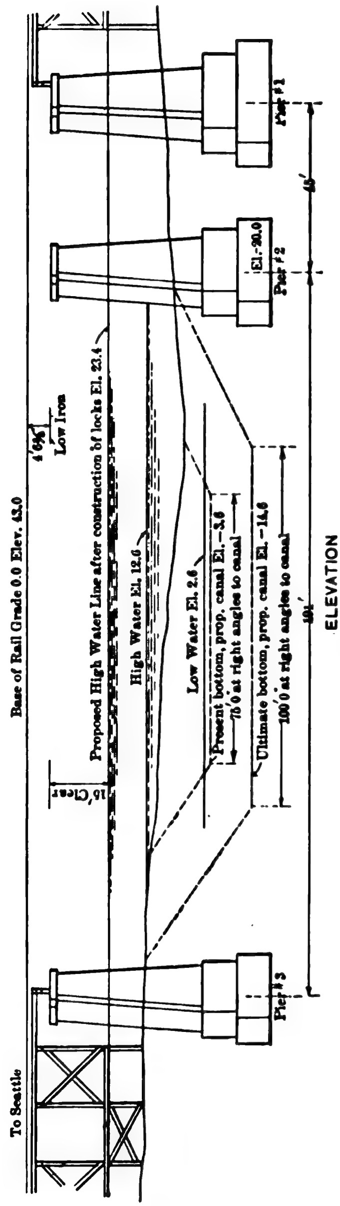


FIG. 88.—SALMON BAY PIERS, NORTHERN PACIFIC RY.

Piers No. 1 and No. 2 on the north side of the canal are 45 feet centers, while pier No. 3 is 191 feet from the center of pier No. 2 and across the canal, as shown in Fig. 88. The borings showed (Fig. 89) a thick layer of wet, soft brown loam beneath the surface; next a layer of soft blue clay, and underlaid by a bed of fine sand carrying 1 per cent. of fine gravel. The plans indicated the probable use of piling under all three piers. The total weight on the foundation of pier No. 1 was calculated to be 6323 tons, on No. 2 to be 4520 tons and on No. 3 to be 3382 tons; giving a pressure on the sand in case piling was not used and taking into account the bouyancy, of 2.98 tons per square foot for No. 1, of 2.81 tons for No. 2, and 2.84 tons for No. 3. In case piling was used as contemplated, the load per pile considering buoyance would

FIG. 80.—SALMON BAY PIERS, N. P. RY. BORINGS.

be for No. 1 pier 21.8 tons, for No. 2 pier 21.0 tons, and for No. 3 pier 21.7 tons. Considering that one ton per square foot of base was carried by the sand, each pile in No. 1 would have to sustain 18.9 tons, No. 2 pier 17.2 tons, and No. 3 pier 19.0 tons.

With the bottoms of the piers at minus 20 and the ground surface near high tide elevation of plus 12.6, and with the soft watertight material to penetrate, the location seemed an ideal one for using sheet-piling for coffer-dams. Accordingly sheet-piles were prepared

FIG. 90.—COFFER-DAM N. P. PIERS.

of 10×12 and 12×12 timber, with dove-tail tongue and groove spiked on (Fig. 91); but upon proceeding to drive them, a hard bottom was struck on pier No. 1 at about elevation −8 to −11, and on pier No. 2 at about −11 to −14. This was found to be so hard, apparently cemented gravel, that it was believed it would stand up while excavating below the bottoms of the sheet-piles, especially if proper lagging or short sheeting was used. Work was continued and all of piers No. 1 and No. 2 were driven, but upon proceeding to excavate pier No. 2, carrying down the bracing as the hole was deepened, the

whole mud flat commenced to slide towards the canal, carrying the tops of Nos. 1 and 2 coffer-dams with it.

Work was then stopped on No. 2 and the decision made to complete No. 1 first to act as a retaining wall to protect No. 2 and upon reaching the bottom of the sheet-piling, it was found impossible to use the lagging on account of the hard material going to pieces when exposed to water. To overcome this difficulty sheeting was driven outside the sheet-piles, of 4×12 plank, 40 feet long, and by this means a depth of -16.5 was reached, where it was decided to found the

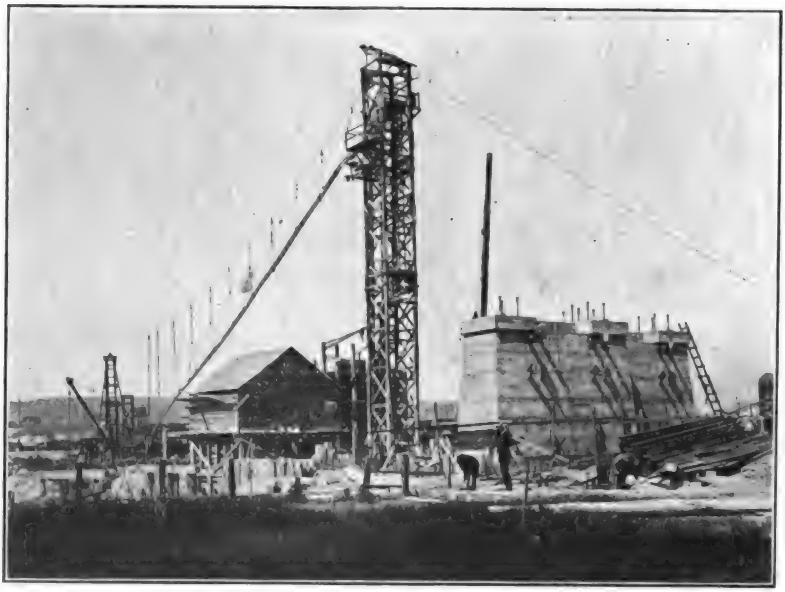


FIG. 90a.—SALMON BAY PIERS, N. P. RY.

pier on the hard bottom without piling. The excavation was all done by a one-yard Owen clamshell bucket, operated on a derrick, from a double $8\frac{1}{2}\times10$ hoist engine, with swinging drums. This bucket dumped into a hopper at the end of a sluice 24 inches wide and 12 inches deep, which was supplied with water to wash away the excavated material, from a 6-inch direct-connected sand pump. This drew its water from inside the coffer-dam, and with the assistance of a No. 2 Emerson pump kept the dam dry. The steam for these and a $7\times4\frac{1}{2}\times10$ jetting pump was supplied by an 80 horsepower vertical boiler.

The material was kept away from the bucket by the sluiceway

described, and some of the material was so slick it would slide in chunks along the sluice, in pieces of a quarter of a yard or more, a distance of about 150 feet to the grade fill back of the bridge.

Upon pumping out No. 2 again and finding that the 4×12 sheeting would have to be driven around it, the decision was reached to complete pier No. 3 next, in order not to disturb the cement house

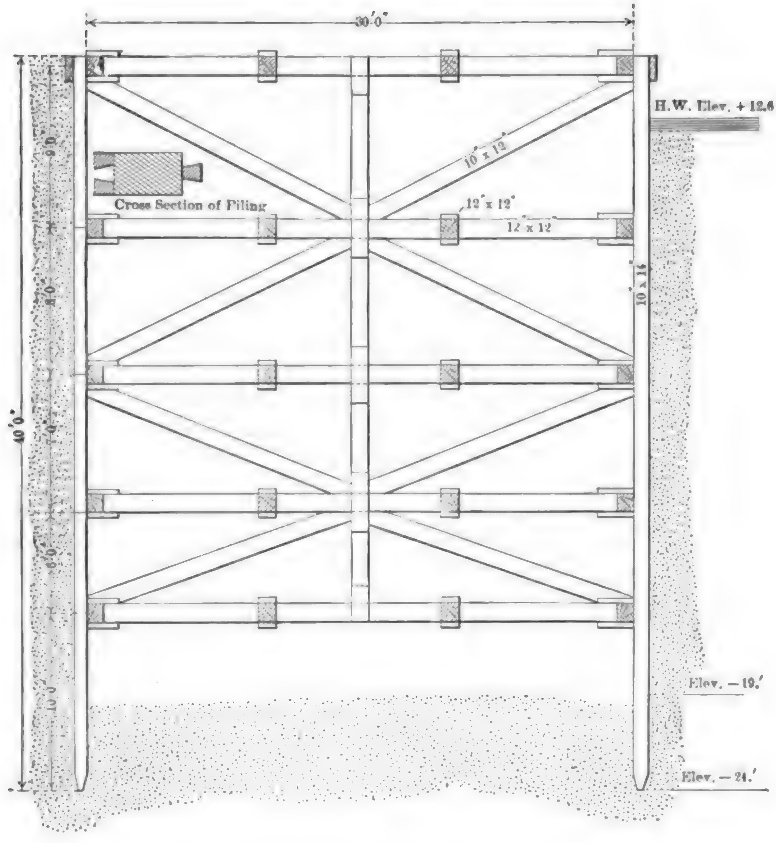


FIG. 91.—SALMON BAY COFFER-DAM PIER NO. 3.

and spouting tower. (Fig. 90*a*.) The sheet piles for No. 3 of 10×12 timber 40 feet long had been driven, and at only this short distance of less than 200 feet from the other piers no hard stuff was struck, and they reached a penetration below −20, the bottom of the pier. This was excavated in the same manner as described for the other piers, the bracing being placed as the hole was dug out (Fig. 91). Bottom was reached without encountering any hard material, but the material

proved to be full of water, which bubbled up through it and made the sand act much as quicksand, swelling up also, so for quite a time no gain in depth was made, and it was decided to drive piling to carry the pier. This was done by means of leads swung from the derrick. The jet was run down into the bottom as described in the chapter on Jetting Piles, the piles then placed and driven home to a penetration of from 25 to 40 feet. The coffer-dam on this pier kept its shape and was easily kept dry by No. 2 and No. 3 Emerson pumps. After the piles had been driven and cut off, the swelled material from the driving was excavated, but as it kept swelling up, it was finally decided to concrete the base at −19. At extreme high spring tides the water came in over the top of the sheet-piles and work had to be stopped for part of each day for some days.

This is a notable example of the successful use of sheet-piling to an exceptional depth, and had the borings panned out no trouble would have been experienced on any of the piers. Had the borings indicated the real nature of the material, metal sheet-piles might have been employed and probably would have penetrated the hard bottom, and the difficulty have been overcome. The lesson to be learned is that any kind of borings, except core borings, are unreliable and are liable to be very misleading.

Upon the completion of pier No. 3, work was resumed on No. 2. The second line of sheeting of 4×12 planking, 40 feet long, was driven and a penetration of several feet more obtained than was realized with the original sheet-piling.

With the exception of a few small breaks during the excavation, which were easily closed by driving the plank deeper, or by inside lagging, no trouble was had, and when elevation −18 was reached, the bottom was leveled off and the concrete base poured. The sheet-piling was removed from all the piers by boring holes into it and shooting off with dynamite, using two sticks to a pile.

CHAPTER VII

CONSTRUCTION WITH SHEET-PILES (CONTINUED)

VARIOUS combinations of the sheet-piling shown in Fig. 52 may be made, when occasion demands, or modifications may be made that will perhaps render the available material more effective. For example, the form (*g*) may be modified to the form shown in Fig. 92, which has the advantage of a wider lap, and should the piles not draw tight together in driving, no crack will be left open to admit the water. Then the piles of this form will act as guides to the ones being driven, similar to the ordinary tongue-and-groove piling.

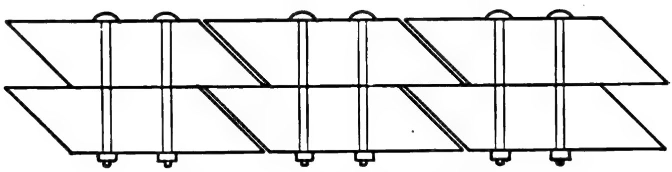


FIG. 92.—COMPOUND SHEET-PILE.

Other combinations and arrangements will readily suggest themselves as necessity may demand.

The use of sheet-piling is often accompanied by a great deal of trouble in securing tightness, and as a matter of precaution, the very best method possible should be adopted in making the piling.

The coffer-dams constructed at Chattanooga for the Walnut Street bridge over the Tennessee River, under Edwin Thacher, consulting engineer, were described in the *Engineering News* of May 16, 1891.

Four piers were founded by this method, but the account of pier No. 2 will fully illustrate the work. The bed-rock, which was level, was covered by cemented sand, gravel, and boulders, of which 320 yards were removed. The coffer-dam was built 18 feet high, or 8 feet above low water, to provide for a future rise. The inside was made large enough to allow of a space of 4 feet all around the base of the pier, and the space between the sheet-piles for a puddle-

chamber was made 9 feet. This was filled to an average of 12 feet with a clay puddle, of which there was 900 yards used. As a protection, there was placed outside the dam about 450 yards of puddle, and a breakwater was built up-stream. About 38,000 feet of timber was used in the dam and breakwater.

After the dam was completed a rise of 30 feet washed out about half the puddle, and one end was crushed by a raft, but the repairs were made without serious trouble. No extra amount of pumping was required on any of this work except pier No. 3, where the seams in the bed-rock required pumps with a capacity of 5000 gallons per minute, and these did not suffice to keep the water down, until the seams were closed by laying sacks of concrete over them and weight-

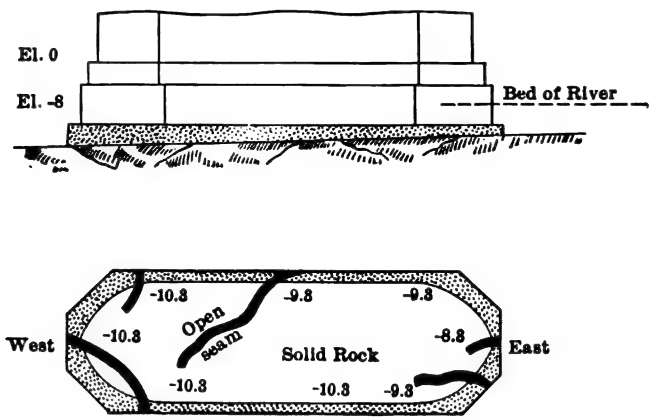


FIG. 93.—CHATTANOOGA BRIDGE, BED-ROCK PIER NO. 3.

ing them down with large stones. The location of these seams is shown in Fig. 93.

The framework and wales for a sheet-pile coffer-dam, used in founding the pier for the Baltimore Street bridge at Cumberland, Md., are shown in Fig. 94, and this was described in the *Engineering News* of July 21, 1892, by H. P. Le Fevre, engineer in charge. The frame was built in place on two canal-boats and after completion was suspended from the old Bollman truss which the new bridge replaced.

The depth of the water was 4 feet, and about 6 feet of very loose quicksand and small round pebbles overlaid the hard bottom.

After the boats were removed, the frame was lowered to its place the sheet-piling driven, and the dam pumped out with a 6-inch pump. The foundation was laid on the hard bottom under the quicksand, after this had been removed.

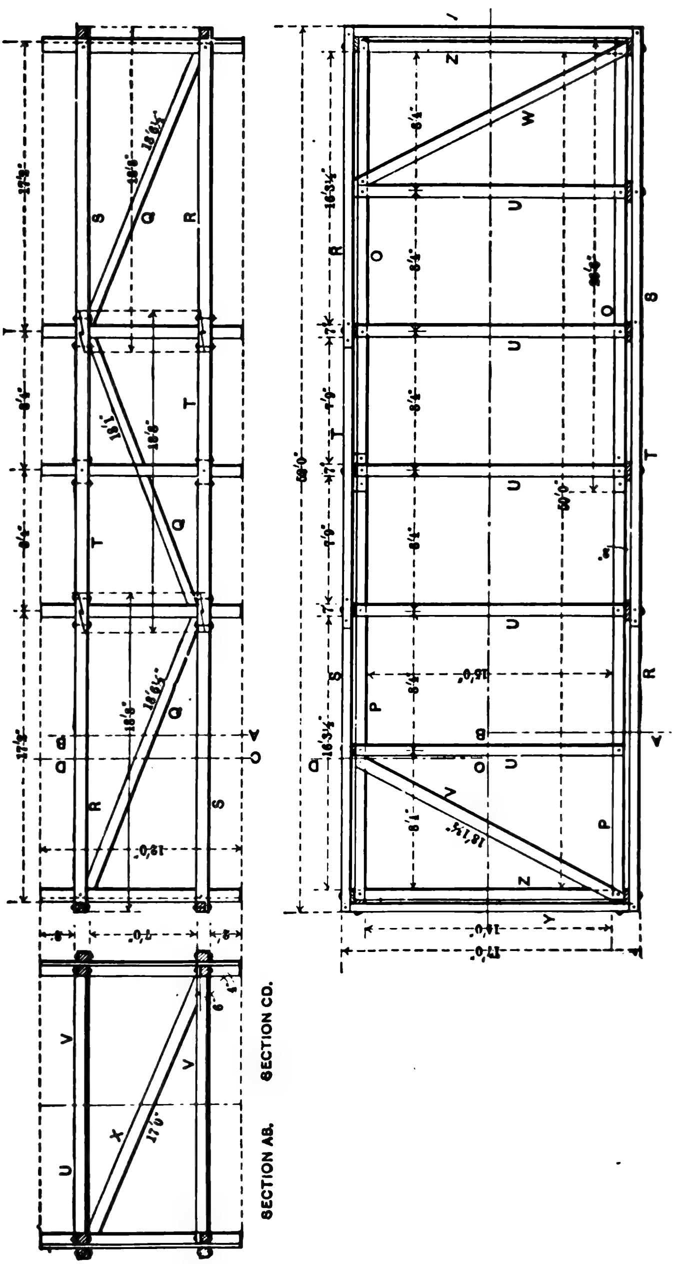


FIG. 94.—FRAMEWORK OF COFFER-DAM, CUMBERLAND, MD.

The grillage was made of two courses of 15×15-inch clear white oak, around which was built a framework, and the open spaces of the grillage were then filled with a concrete made up of one part of Cedar Cliff cement to two parts of sand and four parts of hydraulic limestone broken to pass through a 2-inch ring. Upon this were laid the footing courses of the masonry.

Another ordinary sheet-pile coffer-dam which gave good satisfaction was used at the Sandy Lake dam on the Mississippi River, by Major W. A. Jones, Corps of Engineers, and as the account contains so much of value it will be quoted in full from the 1894 report of the Chief of Engineers.

"The coffer-dam is composed of two rows of round piles, 12 feet from center to center of piles, with the exception of 62 feet of the east end of the upper part, where they were driven 14 feet from center. The piles in each row are 8½ feet from center to center,

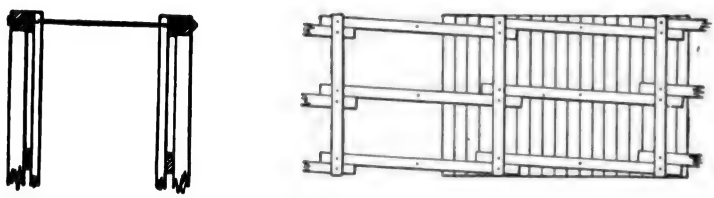


Fig. 95.—Sandy Lake Coffer-dam.

cut off at an elevation of 1217 feet above sea-level and capped with 12×12-inch timber. The inside row of sheeting is 4×12-inch, and the outside 6×12-inch plank. The sheeting is cut off at an elevation of 1218 feet above sea-level, or 2 feet below the flowage line. One-inch rods of round iron, 8½ feet apart, pass through the caps to prevent the filling from spreading the two lines of sheeting at the top.

"In May, 1892, when a flood occurred, the outside of the coffer-dam was raised 3 feet by splicing 3-inch planks to the outside row of sheeting and then filling the triangular prism thus formed with earth. The cross-section of Fig. 95 gives an idea of the dam above the bottom, while the longitudinal section shows the framing down to where it rests on the bottom, the frames being joined by the 1-inch lateral rods of iron.

"The total length of the coffer-dam is 829 feet, of which 742 feet is like that shown in cross-section and the other 87 feet like that shown in the longitudinal section.

"The number of round piles driven in the foundation is 1605. The driving was commenced on November 12, 1891, and completed on August 21, 1893.

"The material in the foundation is sand, excepting in the lower right-hand corner, where there is some blue clay overlying the sand. The sand in the foundation is not as compact as it is usually found in the bed of streams. In the south half of the dam, the surface settled from 6 to 4 inches during the driving. As the surface was settling, the driving became harder all the time. In the north half, which embraces the navigable pass, there was some settlement, but it was not as noticeable as in the south half. The surface had probably settled by the jarring of the hammers while the first half was being driven. The penetration of the piles is also greater than it usually is in sand foundations in the bed of streams.

"The piles were all of Norway pine and well seasoned. Two Mundy steam hoisting-engines were used in driving, one a single-cylinder and the other a double-cylinder engine. In operating the hammer a $\frac{1}{2}$-inch manila rope was attached to the pin connecting the lugs of the hammer, then passed over the sheave at the top of the leaders, and next around the drum of the hoisting-engine.

"When the hammer falls, it pulls the rope with it and unwinds it from the drum. This is what is termed driving with a 'slack line.' The blows are more rapid and keep the material around the piles looser than it would be in the case of using nippers. Iron rings of $\frac{5}{8}\times 2\frac{1}{2}$ inch Norway iron were used to protect the head of the pile.

"It is a well-known fact in pile-driving that it is very important to keep the material from settling around the pile, once it has been loosened, until the pile is down; for when the material has settled, or even partially, the penetration is diminished. The greatest load on a bearing pile is about $13\frac{1}{2}$ tons.

"Sheet-piling was driven by a pile-driver, assisted by a jet of water from a steam force-pump. In driving all sheet-piles a cast-iron cap or follower was used which fitted over the head of the pile. On the upper side of the follower there is a wooden block of some seasoned or close-grained wood which receives the blow of the hammer. This device saves the head of the pile from being battered or splintered, and the pile can be driven to a greater depth than it could be without it.

"In first using the jet on a sheet-pile, a groove was made in the inner edge to receive a $\frac{1}{2}$-inch gas-pipe, which was connected to the force-pump by means of a $1\frac{1}{2}$-inch hose. The aperture at the lower

end of the gas-pipe was reduced to a diameter of about $\frac{3}{8}$ inch. The water was thus forced to the bottom of the pile and the sand loosened.

"This worked well until the sheet-pile struck gravel, when the nozzle of the pipe would become battered or filled with gravel. The pressure in the hose would then burst a coupling somewhere. Another source of trouble was the frequent breakages in the connection between the pipe and the hose, on account of the jarring of the hammer. This plan after a while was abandoned and the nozzle of the pipe was thrust by hand under the point of the pile. The piles are driven in the ground from 12 to 14 feet."

The construction of the Main Street bridge at Little Rock, Ark., involved the construction of two coffer-dams, for piers No. 9 and No. 6. This work was done under the direction of Edwin Thacher, consulting engineer, whose original specifications called for pile foundations for these piers, the piles to be driven to bed-rock and cut off 4 feet below water, to receive a grillage of 12×12-inch timbers to receive the masonry. The size of the grillage being 12 and 13 feet wide by 34 feet long and resting on forty-eight and sixty piles respectively, the piles being of good sound oak or pine at least 7 inches in size at the small end and not less than 12 inches at the butt when sawed off.

The coffer-dams were constructed, as can be seen from the view in Fig. 96, by driving guide-piles, to the top of which are drift-bolted square guide-timbers. The sheet-piling of 3-inch tongue-and-groove stuff was driven against the outside of this timber, and the excavation banked up against the outside. They gave excellent satisfaction and caused little trouble, as the water was shallow.

The piers were constructed of Portland cement concrete, the facing of 2 inches thickness being a mortar of one part cement to two parts of sand, while the balance was of concrete of one part cement, three parts sand, and six parts of broken stone.

Where sheet-piles are to be driven on rock bottom or through earth or gravel to rock bottom, they should be driven hard enough to broom up and form a close joint with the rock. This has been accomplished also by driving the piles with a thin edge until they fit the rock bottom, when they are drawn and after cutting them to conform to the contour of the rock, they are redriven, thus forming a tight joint. This method, while very good, is too expensive for general adoption.

The construction of the piers for the Philadelphia and Reading Railroad bridge over the Schuylkill was accomplished by the use of a floating coffer-dam, the foundations being laid upon the bed-rock.

When in position for work the dam is rectangular in shape, 62 feet long and 36 feet wide, outside dimensions, and 16 feet high. Each side consists of timber crib-work 10 feet wide, making the inside

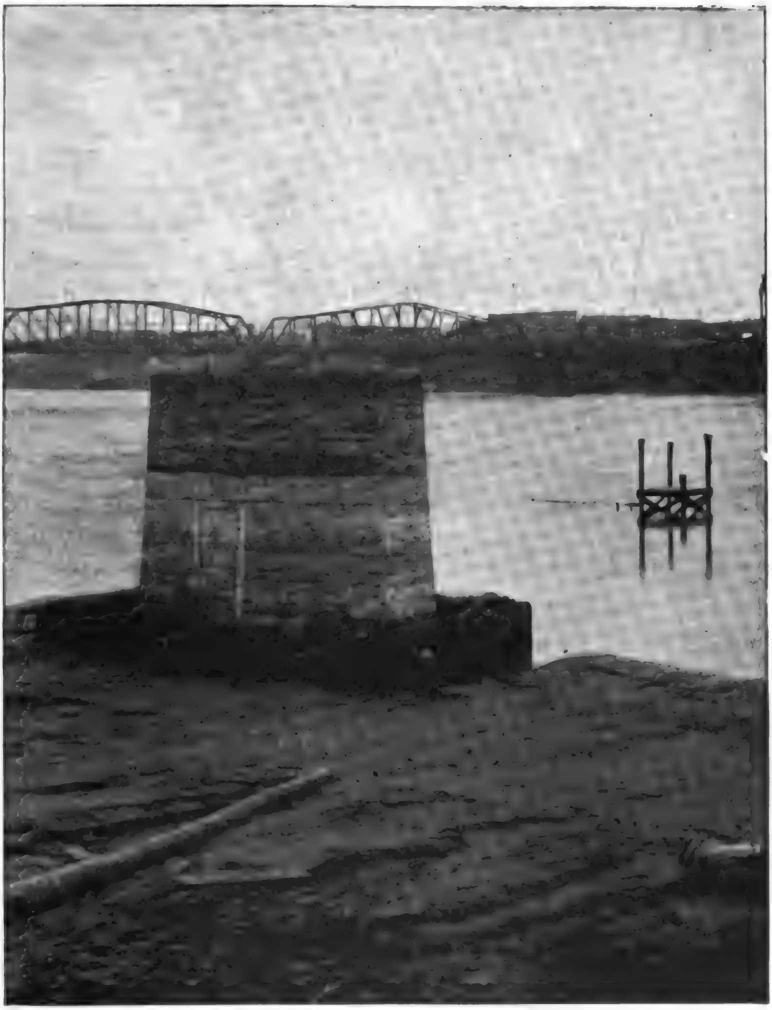


FIG. 96.—COFFER-DAM AND CONCRETE PIER, LITTLE ROCK, ARK.

dimensions 42′×16′. At each corner there is a movable timber extending vertically from the bottom of the crib to some distance above the top. These timbers or spuds are shod with iron on the bottom, and serve to hold the dam in position while the sheet-piling is being driven.

The dam is divided vertically through each short side into two equal parts, which can be floated separately to any desired position and afterwards joined together. Water-tight compartments are also used to hold stone when it is desired to sink the cribs.

When the two sections are united and placed in required position the spuds are dropped and the crib-work is sunk by letting water into the water-tight compartments, and putting in the necessary amount of stone.

Any irregularity in bearing between the bottom rock and the bottom of the crib is then corrected by a diver, who blocks up where required. Close sheet-piling of jointed plank 3 or 4 inches thick is then put on the outside and spiked to the cribs. Puddle, composed of clay and gravel, is then thrown around the bottom outside, and the dam is ready to be pumped out. When the masonry reached the height of the braces they were taken out and the dam was braced against the masonry.

The maximum depth of water encountered at Falls Bridge was 13 feet at ordinary water-level. Several freshets occurred during the progress of the work which did some damage to the dam. At one time, when a dam was ready to be pumped out, a rise in the river moved it down-stream about 30 feet, tearing off the sheet-piling. It was drawn back to place and successfully completed. To make a complete shift of the dam from one pier to the next, with a gang of six men, required about six or eight days, divided as follows: To take the dam apart and reset it, about three days; to sheet-pile, about two days; to puddle, about one day; and pumping out and puddling meanwhile required about one to two days, depending on the amount of the leakage. At each shift, a portion of the plank sheet-piling, perhaps 10 per cent., had to be replaced by new stuff. The pump used was located on a small steamboat, and was run by a steam-engine. The amount of pumping required after the dam was once pumped out varied for the different piers; some dams required little pumping and others a good deal. Only one of the foundations required much leveling off of the river bed, and this one also gave considerable trouble to keep the water out, but the leaks were finally stopped by using gunny bags around them, the bags being drawn into the crevices by the force of the water, thus holding the puddle.

The floating dam was used for three piers in the river channel, the two piers near the shore being put in with ordinary dams. The floating dam is still in good condition and could be used again if needed. The original dam, of which the one used at the Falls Bridge is an enlarged copy, was used for twenty-three or twenty-four settings

The foregoing account is taken from the *Engineering News* of May 24, 1894, the description being by W. B. Riegner, who states also that the cost of the coffer-dam, including one set of sheet-piling, was \$3000, while the total cost for five coffer-dams, including the two crib coffer-dams at the sides of the river, was \$14,000.

The subject of subaqueous foundations has been very fully treated in a series of lectures by W. R. Kinipple, M. Inst, C. E., before the Royal Engineers' Institute at Chatham, England.

The use of 6-inch pitch-pine close sheeting was made use of by him for a quay wall in the harbor of St. Helier, Jersey. They were driven to rock or as deep as possible with a 2800-pound hammer, and the tops cut off a few feet beneath half-tide level, and clayey material banked up against the outside. The bottom through which the sheet-piles were driven was sand and clay.

The rock was laid bare to a depth of as much as 13 feet below low water and in sections which contained about 900 tons of water to be pumped out; this was done with a 16-inch centrifugal pump in about forty-two minutes.

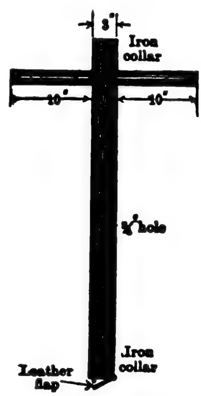


FIG. 97.—STOCK RAMMER.

Several leaks were developed under the piles, but they were promptly stopped by "stock ramming." The stock rammer which is shown in Fig. 97 is 3 inches in diameter, 3½ feet long, and banded top and bottom with iron. A ¾-inch air-hole is bored up from its foot a distance of 20 to 30 inches, and covered on the bottom with a soleleather flap, so that air is let in and suction prevented as it is withdrawn. The sheet-piles have 3¼-inch holes bored through their sides, and cylinders of clay are inserted 3″×9″ long, similar to the work at Sault Ste. Marie. The stock rammer is inserted and driven by mauls as far as its length will permit, when it is drawn out, and other charges inserted until no more clay can be driven, the hole in the pile being filled with a wooden plug.

The piers for the Putney bridge, over the Thames, were built by the same engineer, with single pile-dams to a great depth, by using 14-inch square piles, with elm-wood tongues, and driving them down through the mud and clay to the stiff clay bottom, so that practically water-tight work was secured.

In the construction of the docks at Victoria, British Columbia, he constructed a coffer-dam 500 feet in length, in a depth of 35 feet

of water, the bottom being of rock and overlaid in places with sand and shells several feet in thickness. At the center the sand and shells overlaid a bed of clay.

Three rows of close 12×12-inch sheet-piling were driven with two puddle-chambers of 7 feet each between. The guide-piles were 15×15 inches and the wales were 12×12 inches.

Where the dam rested on rock at the ends, heavy shoes were used on the piles and concrete deposited around their feet to make the work water-tight. This dam was completed in October, 1879, and remained thoroughly tight until the dock was completed over seven years later.

The arch bridge at Topeka, Kansas, over the Kaw River, which is being constructed on the Melan system, of concrete and steel, by Keepers and Thacher, the designing engineers, is a most interesting piece of work. The coffer-dams were required by the specifications to be water-tight, and to effect this 4×12-inch tongue-and-groove sheet-piling was used. The size of the coffer-dam for pier No. 4 was 18×55 feet in the clear (Fig. 98) and the piling was driven about 16 feet into the sand bottom or 22 feet below low water. The driving was done by a 1600-pound hammer with 36 feet leads, the power being furnished by a 15-H.P. hoisting-engine.

No puddle was used around the outside except to stop leaks, and the dam was kept clear of water with a No. 6 Special Van Wie sand-pump. The capacity of the pump was 1500 gallons per minute of water, and from 60 to 80 yards of sand per hour. It was operated with a 15-H.P. engine. The other piers were handled in a similar manner and with no particular trouble.

The growing scarcity of timber will doubtless lead to the exclusive use of metal at some time in the future, to replace sheet-piling for coffer-dams, but where timber is abundant and reasonable care is exercised in its use, it will continue to be of great service in obtaining foundations by this method.

The following account of the construction and failure of the coffer-dam at Dam No. 48 on the Ohio River is taken from the account by J. C. Oakes, Maj. Corps of Engineers, in Professional Memoirs.

"Of the fifty-four dams proposed for the canalization of the Ohio River all of those constructed up to the present time, except No. 37 below Cincinnati and 41 at Louisville, have been constructed in the upper 300 miles of the river above the mouth of the Big Sandy. One dam, No. 29, is now under construction just above Ashland at mile 320, and contracts have been let and coffer-dam constructed for

locks and dams No. 31 at mile 358¼ and No. 48 at mile 804½, or 6 miles below Henderson, Ky.

"All of the dams thus far built or contracted for, except No. 48, have fairly firm foundations, most of them being on rock and a few on gravel. The material at all lock sites above Louisville is supposed to be of a character that will not be easily eroded by the current, so that up to the present time no special precautions have had to be taken for protection of coffer-dams during construction or for protection of the works against undermining by the current after completion and during operation.

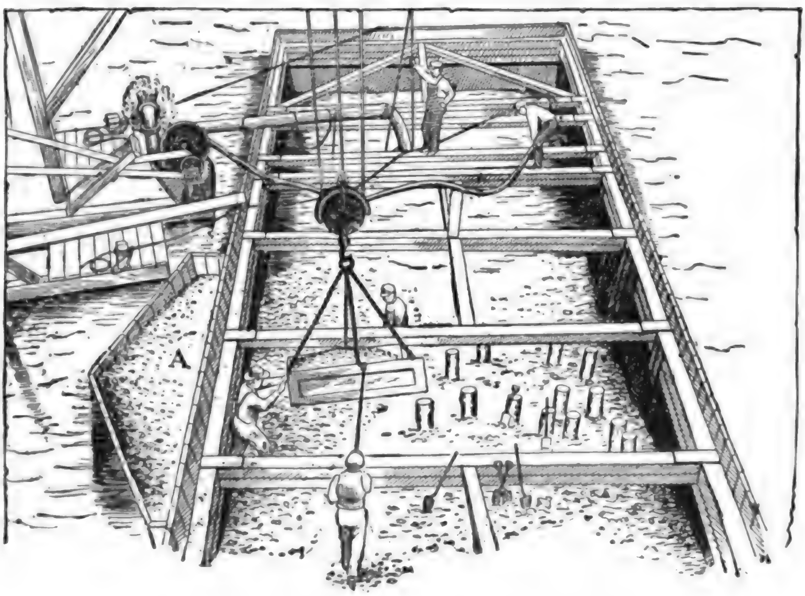


FIG. 98.—TOPEKA BRIDGE, COFFER-DAM NO. 4.
"A" shows puddle to stop leak.

"Below Louisville, however, a totally different type of foundation is encountered. In the thirteen dams to be constructed in the lower 400 miles of the river, rock is found at the sites of only three; elsewhere foundations are fine sand and silt, so fine that the bottom changes with every stage of the river. This fact has caused considerable anxiety, not only with reference to the planning of the works to insure stability after completion, but particularly with reference to the danger to the coffer-dams and erosion of the bed of the river during the period of construction. It has been openly affirmed by some of the contractors who have had experience on the Ohio River

that it would be impossible to construct coffer-dams in the shifting sands of the lower river that would remain during the period of construction, and, second, that if constructed, they could not be made sufficiently impervious against seepage to withstand the ordinary pressure heads, and that they could not be pumped out sufficiently to enable the work to proceed.

"Bids were opened in this office for the construction of No. 48 on September 7, 1911, and it was found that only one bidder had sufficient confidence to offer to do the work. On October 11, 1912. bids were to be opened for the construction of Dam No. 43, but no bids were received. The contract for No. 48 was finally awarded to the only bidder, The Ohio River Contract Company, and during the past season the coffer-dam surrounding the lock, enclosing an area of 20 acres, has been constructed, pumped out, and the round piles under the river wall driven. Work was shut down for the winter on December 31, 1912, and during January, 1913, this work was submerged by one of the worst floods of record at that site, high water reaching an elevation of 371, which is within 2 feet of record high water. No particular damage has been done the coffer and it is expected that it will be in as good condition next season as is usual at other sites after the winter floods. It has therefore been proven that safe coffer-dams can be constructed, maintained, and pumped out without undue trouble at the sites in question as well as in other parts of the river where better foundations exist.*

"It is believed that a short description of the work and type of coffer will be of interest to the members of the Corps of Engineers and also to contractors who might be desirous of bidding on future work in this section of the river.

"At the site selected for dam No. 48, the width between low water lines is 1600 feet; between 20-foot contours, 3100 feet. The lock is to be located on the convex shore of a bend in the river, the radius of which is approximately 4000 feet. Low water is at elevation 325 (Sandy Hook datum), with high water (1884) at 373. The flood of January, 1913, at this site almost equaled that of 1884, reaching an elevation of 371.0. The lock is to be constructed on the Indiana side where the general level of the banks is at elevation 360. On the Kentucky side the bank is somewhat higher, being approximately at 380. The river wall of the lock is to be approximately along the line of low water, which places that wall some 700 feet from the contour of the bank corresponding to that of the elevation of the top of

*See pages immediately following for an account of the failure of this coffer-dam.

the walls. The terreplein between gate recesses is to be connected with the river bank by a causeway or dike at a general elevation of the top of the land wall.

"The lock is the standard size adopted for the river, 600 by 110 feet, with lift of 9 feet. The dam is to have a navigable pass 800 feet wide with Chanoine wickets 16 feet 5 inches long; a Chanoine weir 600 feet wide, wickets 11 feet 9 inches long, and a permanent weir 890 feet long, the elevation of whose crest is to be 1 foot below upper pool, or 337. Upper pool is at 338, lower pool 329, top of river wall 341, and top of land wall 343.

"The material found at the site is a fine sand and silt intimately mixed, with an occasional pocket or very fine gravel. In driving a few long piles to a depth of 47 feet below low water some difficulty was experienced and it is supposed that a layer of gravel was encountered, but no borings have been taken to verify this. This material when quite dry or when entirely submerged in still water stands at about the slope of 2 on 3, but any movement of water either through it or over it rapidly flattens such slope to about 1 in 20. For this reason the coffer-dam was kept 150 feet away from the walls.

"Owing to the distance of the lock from the shore and the necessary contraction of the river, it was thought inadvisable to include any of the navigable pass in the first coffer-dam, which was built therefore to inclose only the lock with arms extending to the bank. The area inclosed is about 20 acres. It is feared that this reduction of the cross-section of the stream will cause considerable erosion, and it may happen that the bed of the river will be so greatly changed as to make it necessary to raise the bed before it will be possible to construct the dam as planned.

"The type of coffer (Fig. 99) is that known as the Ohio River box type, built 20 feet above low water and 20 feet thick for the greater part of its length; it consists of two rows of sheet piles 20 feet apart, tied together by steel rods varying in diameter from $\frac{3}{4}$ inch at the top to $1\frac{1}{4}$ inch at the bottom, held apart by separators and held together by wales on the outside 6 by 6 inches at the top, varying to 10 by 10 inches at the bottom. The space between the rows of sheeting was filled with sand removed from within the coffer by a 10-inch suction dredge, the material being excavated from the lock site. The sides of the coffer were carried into the bank, the top being of uniform elevation 20 feet above low water, To increase the stability of the coffer and decrease seepage a line of 7 by 12-inch triple-lap sheet piles, Wakefield, 26 feet long, was driven outside of and around the coffer and bolted to it.

"No special difficulty occurred during the construction of the coffer, and seepage through the coffer was easily controlled by three 15-inch pumps placed on a pump boat resting on piles near the front of and discharging over the coffer. Some difficulty was met in the extension of the upper arm, owing to the fact that the river rose at a critical period and the pile-driving and placing of skeleton of the coffer was carried on in water as deep as 17 feet. As this coffer-dam extends some 700 feet from the bank at this stage there was, of course, a very serious current around the end, and extraordinary means had to be taken to protect the end of the coffer. Round piles with brush between them weighted down by sand bags and rock were used, and after the corner was turned a pile of riprap was placed to protect it permanently against current and ice. The banking along the front of the coffer was protected by several masses of rock forming short spur dikes to prevent a racing current alongside the dike and the whole length of banking was covered with gravel. This gravel was fine, the largest particles being not more than one-half to three-fourths of an inch in diameter, and some of the material was practically nothing but coarse sand.

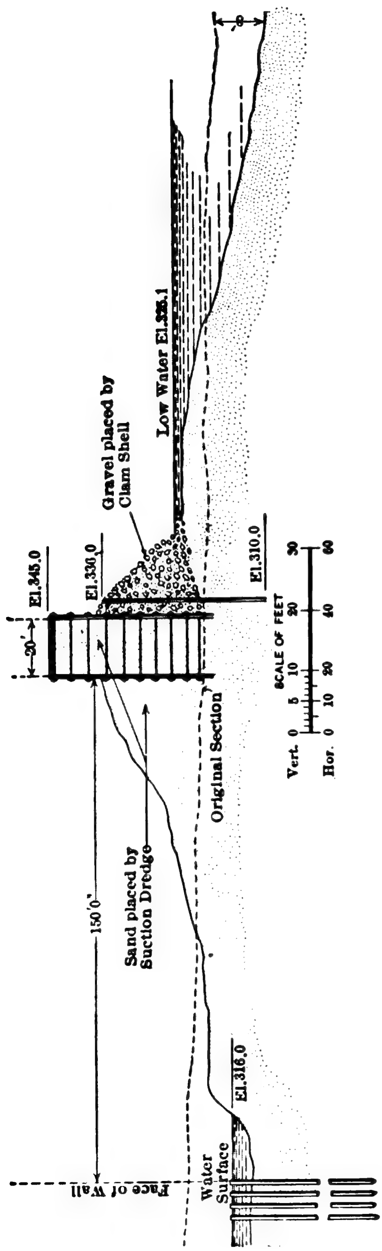


FIG. 99.—TYPICAL SECTION OF COMPLETED COFFER-DAM. DAM NO. 48.

In my opinion, this gravel is much too fine for the purpose intended and the contractor was so notified. He feels, however, confident that it will serve its purpose, and it will be very interesting to see whether his belief is justified.

" The greatest trouble met in the unwatering of the coffer was in protecting the banking against the inside of the coffer from undermining by seepage and drainage water. Wherever there was flowing water the sand was eroded and the banking gradually sloped until it was almost flat. To prevent this, sandbags were used freely, and practically the whole caving surface was covered with the same kind of gravel as that used on the outside. For this protection this gravel was very suitable, and as soon as the sand was covered by a thin layer further erosion did not take place, and the banks remained with a slope of about 2 on 3.

" It is believed that the Wakefield sheet piling driven around the coffer were not used to the best advantage. These piles should have been driven deeper, so as not to overlap the coffer sheeting over 5 feet, instead of about 13 feet as actually driven, and should have been driven close to the coffer. The driving of a great part of the Wakefield piles occurred at medium stages of the river (10 to 17 feet) with the result that the driving was poorly done, and this sheeting is not as tight as it should have been. The contractor was unable to find men who had adequate experience in driving piles in sand with the use of a jet. Both in the case of the sheet-piles and the round piles under the river wall the pile-drivers did not accomplish more than 50 per cent of what they should have accomplished. This was due in part to lack of experience, but also in part to improper pumps, hose, and jets. Up to the end of the season the greatest number of round piles, 30 feet long, driven by one pile-driver in one day was 31. While the jet was used it is very doubtful whether any benefit was obtained, owing to lack of pressure at the nozzle.

" The contractor's attention has been called to these defects, and preparations are being made to provide the pile-drivers with proper jet apparatus, and it is believed that when he begins to drive next season each pile-driver will drive from 80 to 100 piles per day.

" The use of a pump boat with all of the pumps concentrated, the boat resting on piles as the water lowers, has been excellent, and it is believed is very economical and much preferable to other methods commonly used. When the coffer is flooded the boats are simply disconnected from the discharge pipes, their suctions raised, and they are floated out of the coffer through the passway. This furnishes a

very simple and economical manner of removing and placing the pumping apparatus.

"This being the first coffer to be placed on sand foundations in the Ohio River, the contractors have taken very great care to use the best material and to make everything as secure as possible. It is possible that in future coffer-dams some of the measures adopted in this case may be found to be unnecessary and the cost of the coffer-dams materially reduced.

"The following is an extract of a report of the Government inspector in charge of the work, Junior Engineer Edward H. West, in which the work is described in fuller detail: . . .

"The coffer, both inside and out, is heavily banked with sand. A typical cross-section is shown on Fig. 101. Plans were closely followed in construction, the only exceptions being as follows: For a distance of 670 feet from the land end of the up-stream wing and 520 feet from the land end of the down-stream wing, tie-rods were spaced 8 feet on centers instead of 6 feet on centers; in some portions of the coffer, instead of 2 by 12-inch deck joists spaced 24 inches on centers, 2 by 10-inch joists were used, spaced $16\frac{1}{2}$ inches on centers, and in a short portion of the river arm 2 by 10-inch joists 18 inches on centers were used. The sheet piles were driven from a floating pile-driver, and at times the river was too high to permit the piles to be driven to the proper depth; consequently, such piles do not have quite the penetration intended, the maximum difference between actual and intended penetration being about 3 feet; at the up-stream outer corner the length of a number of sheet-piles was increased to 32 feet for additional safety at the point of supposed greatest weakness; at this corner about fifty round piles were driven as shown on Fig. 101; the area inclosed within them was filled with brush weighted with sandbags. Afterward the entire corner was protected with derrick-stone piled as high as the top of the coffer. The lower outer corner was also protected by derrick stone and several stone dikes were built along the river side.

"On June 20, 1912, the line of sheet-piles was commenced, beginning at a point 250 feet from the land end of the upper wing, and piles were driven on fifty-six days, an average of 40 piles per day. On a number of these days, however, two pile-drivers worked, the average number of piles per day for one driver being not more than 30. These sheet-piles were driven rather carelessly, making alignment poor, and leaving the joints not very tight. Better progress could have been made in driving them, but as it was only necessary to keep ahead of the coffer skeleton no great effort was made to push

the pile-driving. After the driving of sheet-piles had been begun, trenches about 2 feet deep and 20 feet apart were dug parallel to the sheet-piles. In these a skeleton was erected consisting of wales and those pieces of sheeting through which the tie-rods passed, the sheeting being driven about 2 feet into the sand. All wales were scarfed for 2 feet at both ends and holes bored for tie-rods through the center of the scarf. Where the spacing of tie-rods was 8 feet 18-foot timbers were used for wales, and where the spacing was 6 feet 20-foot wales were used, thus allowing a lap of 2 feet at each end. At each tie-rod a temporary separator perpendicular to the wales and 20 feet long was placed and the nuts on the rod tightened. The remainder of the sheeting was then driven and after the proper cut-off

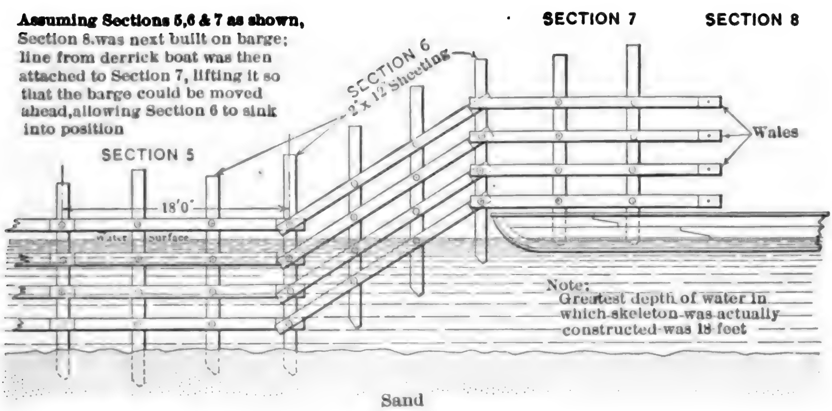


FIG. 100.—METHOD OF EXTENDING 20-FOOT COFFER-DAM. LOCK AND DAM NO. 48.

elevation had been marked the ribbing strips were spiked on and the sheeting cut off to grade. Cracks between adjacent pieces of sheeting were closed by 1×3-inch battens nailed on the inside. When the coffer had been extended into water so deep as to prevent further work from land the skeleton was bolted together on a small barge, one section at a time, and as each section was completed it was lifted by a derrick boat until the barge could be moved forward, when it was lowered into the water (Fig. 100). Sheeting was then driven by men standing on the wales. Throughout the construction two gangs of carpenters worked; at times each gang extended both skeleton and sheeting, but better progress was made when one gang worked on skeleton only and the other followed up with sheeting. Each gang contained at different times from ten to twenty men; it was found that not more than twenty men could work together to

advantage. In general, it may be said that the rate of progress depended upon the rapidity with which the skeleton was advanced, no difficulty being experienced in keeping close up with the other work. As a matter of fact, sheeting was frequently delayed in order that the skeleton might be extended. During the season the skeleton was actually extended on about seventy-one days, an average of 38 feet per day; this value corresponds to two wale lengths, either 36 or 40 feet, depending on the length of the wales being used; it was noted in the field that two wale lengths constituted an average day's work. Sheeting was actually driven on fifty-three days, an average of 52 feet per day (double row) from a minimum of 10 feet to a maximum of 103 feet, but on a great many days on which sheeting was driven much other work was done by the same gang and the above values are not reliable for progress estimates. It is believed that one gang can extend a double row of sheeting 100 feet per day. The extension of skeleton may be assumed to be work accomplished by a single shift, since night work on this portion of the coffer is negligible; sheeting, however, was driven quite satisfactorily at night. After the sheeting had been cut off to proper elevation, the deck joists were laid and spiked to the ribbing strips. This having been done, the discharge pipe of the 10-inch suction dredge was moved into position and the sheeted portion of the coffer filled with sand taken, if possible, from the "pay excavation" for the lock walls. Bulkheads across the coffer were built at convenient intervals as the construction advanced. As the coffer filled, the separators were removed and afterward used again. At first, a long trough built on top of the deck joists of rough lumber, on a grade of 1 per cent., was used for distributing the sand, but it proved to be very inefficient and was discarded; subsequently the desired distribution was accomplished by moving the end of the discharge pipe. Short time tests of the suction dredge showed that it could place 100 cubic yards of fill per hour, through 300 feet of discharge pipe, maximum lift 20 feet. It handled a total of about 100,000 cubic yards of material. The coffer contains only about 40,500 cubic yards; the remainder was used for banking. Of the total excavation made, about 50 per cent. was within the specified limits of "pay excavation." As shown on Fig. 101, a flooding sluice was built in the lower wing and a passway for removing boats, etc., was also left in the lower wing. Discharge sluices and coal tracks were constructed in the river side near the crest line of the dam. A layer of fine gravel was spread over the whole river side of the coffer-banking to prevent scour as much as possible. It is believed

that the one flooding sluice will be all that is necessary, unless very extraordinary conditions arise; in flooding the coffer when the river is near its top and too great a volume of water entering would endanger the permanent work, it is intended to place blocks behind alternate needles, as is often done in regulating pools above needle dams. In the event that this scheme should fail to pass a sufficient quantity of water to fill the coffer before it is topped by a rise, it is intended to use the passway as an emergency opening after constructing a temporary sluiceway.

" The tongue-pieces for sheet-piles were dressed in the contractor's planing mill, as was all material requiring finish, such as sheeting for sluiceways, needles for closing sluices and passway, etc. Two piledrivers were used during the season, both mounted on decked barges, with cabins. One barge was $20 \times 50 \times 4$ feet (No. 30), drawing 25 inches, and the other (No. 29) $22 \times 60 \times 5$ feet, drawing 25 inches. On No. 30 was one 40-horse-power locomotive-firebox portable boiler, made by the Brownell Company, Dayton, Ohio; one two-drum hoisting engine with four winch-heads, made by the American Hoist and Derrick Company, cylinder diameter 7 inches, length of stroke 10 inches, rated horse-power 20; the hammer was a No. 2 steam-hammer, made by the Vulcan Iron Works, Chicago, weight of moving parts 3000 pounds, gross weight 6500 pounds, strokes per minute 60; the make or size of the pump is not known, but its indicated pressure varied from 80 to 110 pounds; the pressure at the jet is unknown, but pressure loss between pump and jet was considerable; the jet was a 2-inch pipe reduced to $1\frac{1}{4}$ inches at the nozzle. On No. 29 was one 60-horse-power locomotive-firebox portable boiler, made by the Brownell Company; one nondescript engine on Lidgerwood base with two winch heads, cylinder diameter $6\frac{1}{4}$ inches; length of stroke, 8 inches; horse-power, 12. The hammer was a No. 3 steam hammer made by the Vulcan Iron Works; weight of moving parts, 1800 pounds; gross weight, 3800 pounds; number of strokes per minute, 60; a McGowan pump supplied water to the jet at about 90 pounds pressure, as indicated at pump, but it is believed that the greater part of this was lost before reaching the nozzle; the jet was a 2-inch pipe, reduced at nozzle to $1\frac{1}{4}$ inches. At different times, three derrick boats were used; two of these were identical, having been recently built for this contract; the third, No. 31, was an old boat, quite inefficient and nearly worn out. Nos. 33 and 34 were built on hulls $34 \times 66 \times 4$ feet 9 inches, drawing about 20 inches; the cabins are 28×28 feet. The one boiler on each boat is a locomotive-firebox portable boiler, 40-horse-power, made by the

Houston-Stanwood & Gamble Company. of Cincinnati; each engine a 50-horse-power three-drum tandem engine for derrick boats, built by the Lidgerwood Manufacturing Company, cylinder diameter, 10 inches; length of stroke, 12 inches; drums, 16×24 inches; the swinging engine is a 6-horse-power Lidgerwood, cylinder diameter 5 inches, length of stroke 6 inches. The fittings were made by the American Hoist and Derrick Company. The boom is 65 feet long and the mast 36 feet. Each boat has three spuds. All excavation, banking, and filling was done by a suction dredge; hull, 22×100×5 feet 6 inches, drawing 24 inches, with one spud at the suction end. The two boilers are each 60-horse-power, built by the Houston-Stanwood & Gamble Company; one engine, 100-horse-power, built by the same company; cylinder diameter, 15 inches; length of stroke, 20 inches. The pump is a 10-inch centrifugal sand-pump, belt driven, made by the Morris Machine Works, Baldwinsville. N. Y., with 10-inch suction and 12-inch discharge pipes. For pumping out the coffer two-pump boats were available; one, built especially for the purpose, contains, on a hull 36×106×5 feet 6 inches, three 100-horse-power boilers, made by the Houston-Stanwood & Gamble Company; three 100-horse-power engines built by the same company; cylinder diameter, 15 inches; length of stroke, 20 inches; and three 15-inch centrifugal pumps, belt driven, made by the Morris Machine Works, and having 18-inch suction and 15-inch discharge pipes. The boilers consumed 330 bushels of coal per twenty-four hours during a run of eighteen days, and each pump, discharging 26 feet above water surface, handled 8000 gallons per minute. The old pump boat has never been used on this contract; it contains two Brownell boilers, 60-horse-power each; one engine, made by Charles Barnes & Company, Cincinnati, cylinder diameter, 15 inches; length of stroke, 18 inches; one engine, built by the Nagle Engine an Boiler Works, Erie, Pa., cylinder diameter, 12 inches; length of stroke, 16 inches. Both centrifugal pumps were built by the Morris Machine Works, Baldwinsville, N. Y.; one is a 15-inch pump and the other an 8-inch sand pump, both belt-driven. The floating machine shop contains a lathe made by the Hamilton Machine Tool Co., Hamilton, Ohio; a pipe-threading machine made by the Merrell Manufacturing Company, Toledo, Ohio; a pipe-threading machine and a drill-press made by Davis & Egan, Cincinnati, Ohio; and a hand-made forge.

"The coffer-dam was closed November 12, 1912, and a typical cross-section of river side is shown on Fig. 101. Inside were left both pump boats; the old derrick boat for use as a clam-shell dredge;

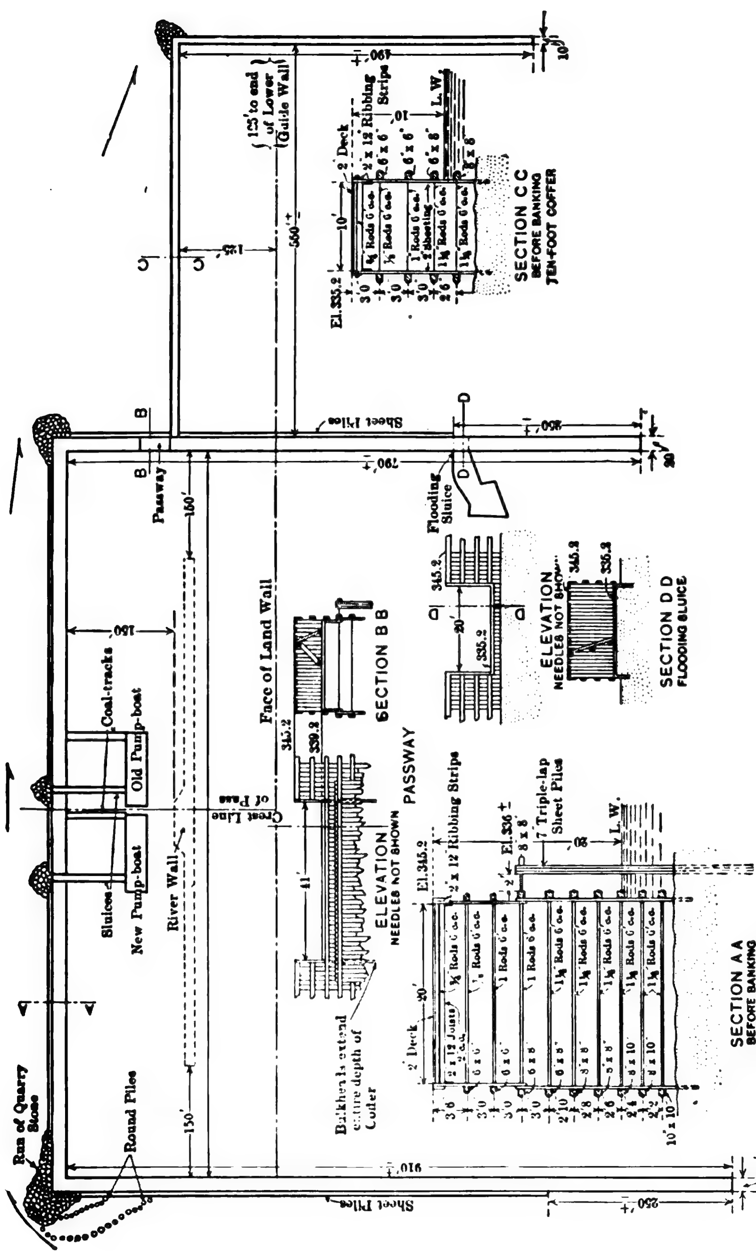


FIG. 101.—COFFER-DAM, OHIO RIVER. LOCK AND DAM NO. 48.

and both pile-drivers. A temporary sluiceway was constructed, discharging through the passway, and a coal track was built to supply fuel to the new pump boat *Uncle Joe*, which was temporarily located over the hole excavated for the lower gate track and recess. It was intended to pump down low enough to enable the pile-drivers to drive 126 piles for supporting the pump boats in their permanent locations. Pumping was begun on Friday, November 15, 1912, at 2 P.M. and was continued until Saturday, November 16, 1912, at 3:10 P.M., when the mouth of the discharge sluice was covered by the rising river and it became necessary to stop the pumps. On Thursday, November 21, the river had fallen sufficiently to begin pumping and the pumps were started at 9:15 A.M. and ran continuously until the piles for pump-boat foundations were driven and capped. Gages had been set inside and outside the coffer and an inspector was kept on the pump boat continuously during the first pumping. The slope of the discharge water was measured in the sluiceway by a series of small gages set before pumping commenced, and the actual discharge while the pumps were making 350 revolutions per minute was computed by Kutter's formula, using a coefficient of rugosity of .009, and found to be 24,000 gallons per minute for three pumps. An attempt was made to obtain more or less accurate estimates of the seepage through the coffer-dam at different heads. Since the pump-boat was floating the vertical distance from water surface to center line of discharge pipes did not vary and the discharge was assumed as constant at 480,000 gallons per pump-hour as measured. The inside gage was read as closely as was possible (to hundredths of a foot) and recorded hourly and a careful record kept of the actual number of pump-hours run. The outside gage was read at 8 A.M., 12 M., and 4 P.M., and the river stage at other hours could readily be estimated. From the readings of these two gages the head on the coffer at any time could be computed. A careful survey had been made of the interior of the coffer by soundings on October 31; from the data obtained by this survey a contour map was plotted on a scale of 1 inch equal 40 feet, contour interval 1 foot, and areas were measured by planimeter at each foot of elevation through the probable range of water surface during pumping. The area of water surface at each hour was obtained by interpolation and the amount by which the water content of the coffer was reduced was computed by average end areas; the seepage during each hour was taken as the difference between the amount of water handled by the pumps and the amount by which the coffer content was reduced. For example:

Date: November 22, 1912.

Time P.M.	Outside Gage.	Inside Gage.	Head.	Pump-hours run.	Water actually pumped.	Content Reduced.	Seepage.	Seepage. per minute.
7	332.53	319.79	12.7					
				2.00	960,000	144,610	815,390	13,589
8	332.50	319.68	12.8					
				2.00	960,000	259,870	700,130	11,669*
9	332.48	319.48	13.0					
				2.00	960,000	230,510	729,490	12,158†
10	332.45	319.30	13.2					
				2.00	960,000	252,380	707,620	11,794
11	332.43	319.10	13.3					

* Average Head = 13.0.
† Seepage per Minute = 12,052.

"It was impossible to read the inside gage closely enough to make the seepage estimates for each hour agree perfectly, and so in plotting the curve shown (on bottom, Fig. 102) the points plotted are average values for several successive hours. While conditions were such that an absolutely accurate estimate of seepage is impossible, the values as given are believed to be close enough for all practical purposes. It would seem that the seepage should vary as the square root of the head, following the well-known laws for flow through pipes and orifices, the passage of the water through sand amounting to flow through an infinite number of minute pipes, but the values actually obtained as outlined above indicate a straight-line variation, that is, directly as the head (top of Fig. 102).

"It will be noticed that from a head of 13 feet to 15 feet the seepage appears to rapidly decrease and from 15 feet to 18 feet it appears to increase more rapidly than normal. This phenomenon occurred during the night when no attempt was being made to lower the water surface rapidly. One pump had been cut out and the other two speeded up so that they were pumping more than the quantity of water measured at normal speed, but how much more is not known; consequently, the seepage appears to decrease, but in reality such is not the case. After the piles for the pump-boat foundations had been driven and capped the coffer was allowed to fill, the boats were floated into position and the *Uncle Joe* pumped down, allowing them to settle on their foundations. Pumping was continued until the water surface inside reached an elevation of 310.7, when the coffer was accepted as satisfactory; it was then allowed to fill up to 316.0, and driving of round piles for river wall foundation was begun from floating drivers.

"Seepage was apparently uniform around the entire coffer perimeter, since a single large leak would have resulted in great damage and the wrecking of a portion of the coffer-dam. Naturally, this seepage water collected into a relatively small number of rather

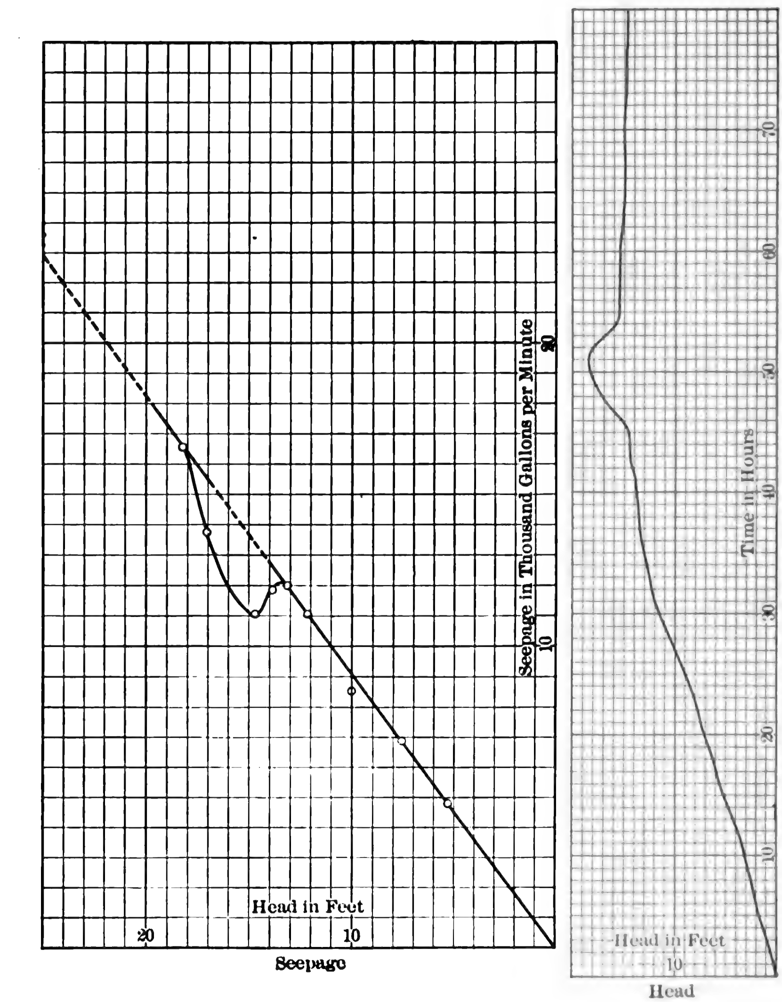


FIG. 102.—SEEPAGE THROUGH 20-FOOT LOCK COFFER. DAM NO. 48.

large streams, moving toward the pumps, each of which carried a large quantity of sand, which was deposited in the still water where the pile-drivers were floating. Taking care of the seepage was a simple matter, but serious difficulty was encountered in preventing the

excavation from filling with this sand. The clam-shell dredge proved to be an inefficient and costly method of solving the problem.

"On several occasions pile-driving was suspended until sand thus washed in could be removed. It was thought that by building dams of sand bags across those streams the movement of sand could be stopped. This was done and proved to be very advantageous, but when water began to flow over the dams sand continued to move into the excavation, though in diminished quantities. A further diminution in the sand movement was effected by spreading a 6-inch layer of gravel over the greater part of the river side of the interior banking. In spite of all efforts, however, large quantities of sand continued to move into the 'hole' and at the time work was suspended for the winter the problem of driving piles without constant excavating was still unsolved. . . .

"At about 7 A.M. Monday, July 21, 1913, there occurred a failure of the main coffer-dam at lock and dam No. 48, Ohio River. This coffer was built 20 feet above low water and at the time of failure the river stage was 12.4 feet above low water, or elevation 337.5 (Sandy Hook datum). The water surface within the coffer was at elevation 316.5, the head being 21 feet.

"The failure occurred at the passway in the lower arm. This passway was left to enable the contractor to pass floating plant in and out of the coffer at stages from 18 to 20 feet, and, as originally constructed, was 41 feet wide with bottom 6 feet below top of coffer, the area being closed by needles, as shown on the illustration.

"After work had closed for the winter and the Government inspector withdrawn, this passway was torn out to remove some floating plant, and without the knowledge of this office was rebuilt with the top 2 feet lower than before. The top of the coffer, therefore, at this passway was only 12 feet above low water, needles being used to close the opening when required. On the outside of the coffer was a line of Wakefield sheet piles 26 feet long, with tops of piles from 10 to 12 feet above low water. These sheet-piles extended about 10 feet below the bottom of the coffer. Sand was banked against the coffer to the height of the tops of the sheet piles.

"In order to hold the banking against the coffer on the inside, the contractors had driven a line of sheet-piles 60 or 65 feet away from the coffer with tops of piles at about low-water elevation and fill had been placed sloping from the tops of these piles to the coffer at the elevation of the floor of the passway. This fill had been covered by gravel to prevent wash by seepage.

" A small quantity of water, probably 1 cubic foot per second, has always collected behind the passway and flowed as a little stream into the excavated area, but as there were may of these little streams, some of them greater than this one, no fear of failure because of this seepage had been felt.

" The Government inspectors lived on a quarterboat moored just below this passway and crossed the coffer constantly at this point, and no increase in the amount of seepage nor any movement of sand had been noticed.

" The excavation was kept clear of water by three 15-inch centrifugal pumps on a pump boat, the latter resting on piles. The engineer of the pump boat noticed nothing unusual until 5:30 A.M. July 21, when he found the pumps were not holding the water surface at 315.5. He then changed his governors, increasing the speed of the engines and finally cut out the governors entirely, allowing the engines to run full speed. In spite of this, the water surface inside rose to 316.5 at 6:30 A.M. Shortly after this the men coming to work noticed a large leak near the passway; the alarm was given, and the pumps stopped at 6:45 A.M. At 7:01 A.M. there was evidently a blow-out, as the water was seen to suck down outside of the sheet-piles; the sheet-piles were lifted out, then the coffer lifted, and the break was completed. Before the inclosure was filled, about 250 feet of coffer had been washed away.

" Through the gap thus created there were drawn four loaded coal barges, a barge of lumber, and one of round piles. The coal barges were rolled over and over, and were a total wreck, the pile barge was broken up, and the lumber barge was injured, but can be recovered and repaired. The pump-boat was thrown off its pile foundation and somewhat injured and four pile-drivers were submerged and probably injured to some extent, the amount of injury being unknown.

" The contractor was about ready to place concrete for the river wall. The excavation was completed, all round piles and most of the sheet-piles were driven and tracks on piles for derricks, cars, etc., were completed. A large amount of sand has been carried into the excavation, coal has been dumped about the heads of the piles and undoubtedly some of the tracks have been injured. It is estimated that the immediate money loss to the contractor will amount to between $10,000 and $15,000, but the accident may cause a much greater loss due to the delay, which is estimated at a month or six weeks, which may prevent the completion of the work inside the coffer this season and make necessary the unwatering again next year.

It is hoped, however, that further investigation when the river falls will show less damage than is anticipated.

"Several causes of the failure can be suggested, and it is probable that they all had a bearing; their relative importance, however, can only be guessed at. These probable causes are: (*a*) The weight of the passway coffer was probably about 1000 pounds per square foot less than it would have been if it had been built to full height; (*b*) in rebuilding the passway, good connection between the old and new sheet-piles and the sheeting of the coffer may not have been obtained; (*c*) the material used for filling the passway coffer when rebuilt may have contained a large proportion of silt as the silt deposit in the coffer during the winter was very heavy; (*d*) the seepage probably gradually increased with the increasing head until, during the night, the little stream began to carry out sand from beneath the coffer and it also probably cut down into the inside banking until the limiting plane of saturation was reached when the blow-out occurred.

"It is certain that there was a considerable increase of flow during the night from this and probably other seepage streams, and that at 6:30 A.M. there was a bad leak under the passway, and when the blowout took place the whole mass became suddenly fluid and lifted the sheet piles and the coffer."

The troubles described in the preceding pages are often the result of not properly fixing the responsibility for the design of cofferdams or other temporary works. When the contractor is made responsible, the plans are often required to be submitted for approval and changes are frequently made in the design that causes failure. When the plans are furnished the contractor, the changes suggested by him are too often disregarded, even though they are meritorious, and troubles ensue which often result in delay, damage, or litigation.

The most frequent cause of failure is the adoption of the wrong method, such as the use of sheet piling where cribs should have been used, or of cribs where sheet piling should have been employed. When sheet piling is to be used, it should be of sufficient thickness to stand the pressure, and be driven to a great enough depth to prevent leakage underneath. The dove-tail tongue and groove as shown in Fig. 91 is one of the best types to use, to avoid leakage between the sheet piles, but if the piles must be driven very hard jets should be used, or else steel sheet piling employed if the cost is not prohibitive.

The fact that there have been many cases of trouble with large sheet pile coffer-dams, such as is referred to on page 25 under conditions of work in the Ohio river very similar to the work described

in this chapter for Ohio river dam No. 48, indicates that some other type of construction should be adopted.

The method used for the dams in the Great Kanawha river, of log cribs, is one that is usually successful and should be very carefully considered in locations where timber is available at reasonable cost. The work for the Tacoma water system crossing of Green river, described on page 36, was carried out in this manner and was a success, even though a change in the type of permanent construction made the space enclosed somewhat small for the operations necessary for placing reinforcing, forms, and concrete.

CHAPTER VIII

REMOVING OLD PIERS

COFFER-DAMS are quite frequently constructed for the repair or removal of existing piers. A pier which was constructed in 1840 in the river Farnitz, at Stettin, Germany, became an obstruction to navigation and it was decided to remove it. The work was described in the *Engineering News* of July 14, 1892.

Its exterior showed a facing of granite laid in hard Roman cement, and soundings revealed the existence of a course of sheet-piling around the pier, with a protection of riprap at its foot. The original drawing of the pier showed a pile foundation. The specification prescribed the use of the old course of sheet-piling shown at *a*, Fig. 103, for the construction of the coffer-dam. Owing to the belief that the existing sheet-piling, after having served such a length of time, would not be sound enough to permit of its use in the erection of a coffer-dam, local contractors could not be found and the work was let to an outside contractor.

The preliminary work was begun by picking up the riprap around the foot of the pier with a claw dredger mounted on a raft. Some of the stones weighed as much as a ton. The bottom of the river, after the riprap had been cleared away, was found to be covered with a layer of concrete, consisting of pieces of brick and cement. This was brought up in large slabs. The pier itself was found to be of rubble masonry, composed of irregularly-shaped granite blocks with the interstices filled with brick, laid in cement mortar. The single stones were detached and swung off by the claws of the dredger. Their average weight was about $1\frac{1}{2}$ tons.

After the masonry had been pulled down to nearly the level of the water a row of sheet-piling, shown at *b* in Fig. 103, consisting of piles 7 inches thick, was driven to a depth of nearly 10 feet. The space between the old and new sheet-piling was filled with new clay. To keep the interior free from water two pumps were employed. After putting in the necessary bracing the work of removing the masonry to the bed of the river was continued. A shell of the latter,

however, was left standing. Then the timber platform on which the masonry had been resting and the layer of concrete below were taken out, exposing a layer of clay underneath. While attempting to pull one of the foundation piles a stream of water rushed through the opening thus formed, so that this plan had to be given up and blasting resorted to. To do this the tops of the piles were bored to a

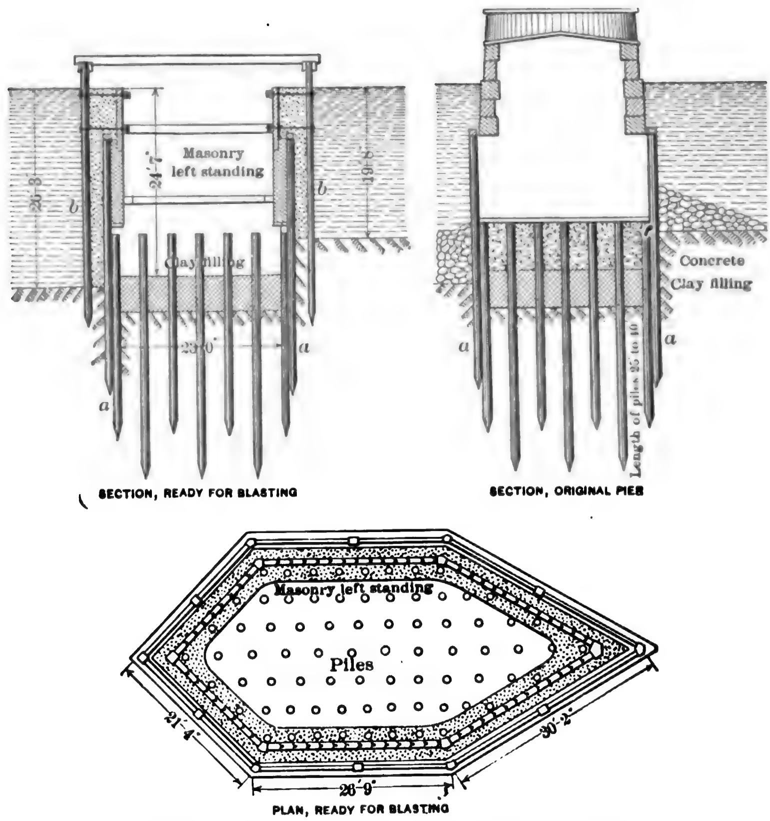


FIG. 103.—REMOVAL OF MASONRY PIER AT STETTIN, GERMANY.

depth of 13 feet and filled with 8.8 pounds of dynamite each. The initial charges consisted of 10.6 ounces in air-tight canisters. The shell of masonry left standing received four cubical charges of 8.8 pounds each. In all sixty-eight charges, consisting of 616 pounds of dynamite, were used. The electric current for the blast was divided into three currents, each being attached to an induction apparatus. The blasting, however, did not prove to be as effective as was antic-

ipated, owing to the dissolving action of the water, and several charges were taken out intact. The clearing away of the wreck was almost entirely done by the claw dredges. The piles, which were split and loosened in their sockets by the force of the explosion, were pulled up by windlasses mounted on flatboats. The work of removing the pier lasted nearly nine months and the cost was about $8700.

Another example of the removal of a pier was at Gadsden, Ala., where a pivot pier in the Coosa River had tilted. The pier had been built originally in a water-tight caisson and was supposed to have been founded on solid rock, but by some error a layer of gravel was left underneath and eventually the pier tilted down-stream 7 feet, nearly throwing the swing span into the river.

After the span had been blocked up to allow the passage of trains, a coffer-dam was built around the pier to give plenty of clearance to the old caisson. (Fig. 104.) This was constructed by driving three rows of sheet-piling through sand and gravel to bed-rock and puddling between them.

The sand and gravel over the rock were not removed from the bottom of the puddle-chamber before puddling and a great deal of trouble was experienced all through the work by leakage through the porous gravel. It is probable, too, that a poor joint was made between the sheet-piling and the rock.

Bents were erected upon the sides of the coffer-dam and by driving piles into the puddle and inside the dam, to carry a truss on each side of the span, which carried the drum and supported the main trusses at the center. When this had been tested by loading with trains of ore upon the bridge and found to be satisfactory, work was at once begun upon the removal of the old pier, by means of two fixed derricks on the false work and one floating derrick. The stones were marked as they were removed to insure their return to proper places when the pier was rebuilt, and was taken to the shore until needed again. When the masonry was all removed the grillage was broken up and taken out, after which the gravel inside the coffer-dam was cleaned out down to bed-rock. New footing courses were laid to take the place of the gravel and old grillage, and the old stonework relaid by placing each course in its former position as nearly as possible. The pier was about 80 feet high and contained about 1100 yards of masonry. The work occupied from Sept. 15 to Dec. 25, 1888, and was done under the direction of Cecil Frazer. The description is taken from the *Engineering News* of April 13, 1893.

The reconstruction of the center span of the central viaduct

across the Cuyahoga River, Cleveland, involved the elimination of the old low-level drawspan and the substitution of a high-level fixed span, as described in the *Engineering Record* of July 6, 1912. The expensive old shore-piers were retained and their foundations

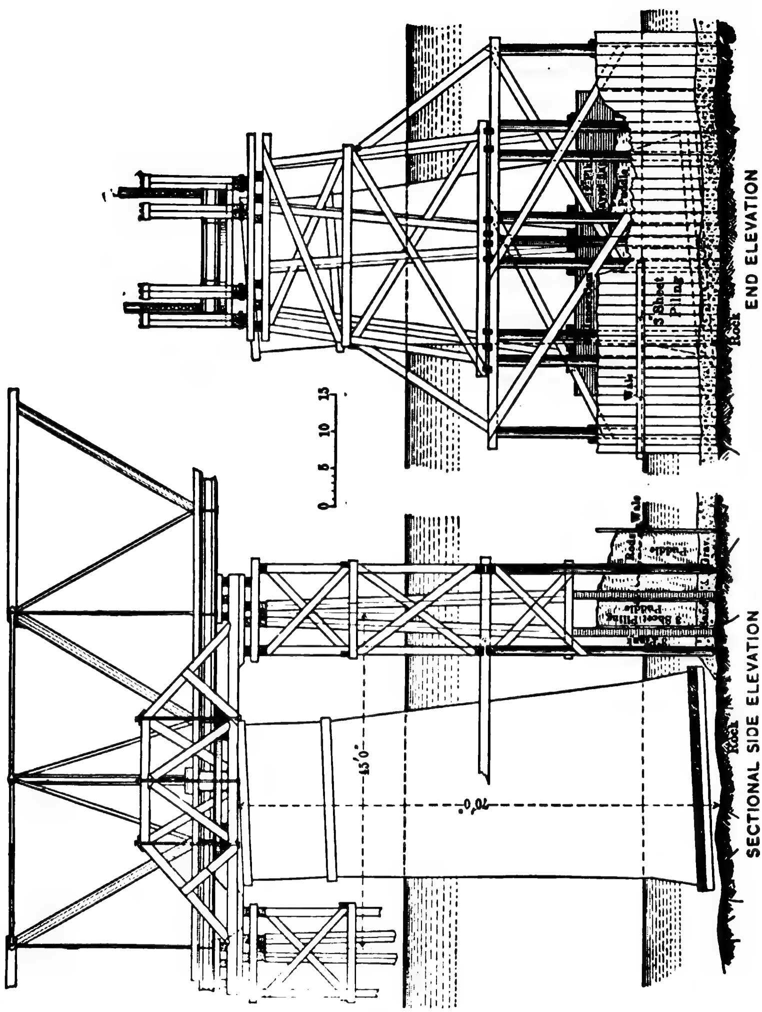


FIG. 104.—COOSA RIVER COFFER-DAM.

were reinforced and extended as described in the *Engineering Record* of June 29, 1912. The improvement also involved the removal of the old pivot pier by the Great Lakes Dredge & Dock Company, which was the contractor for the reinforcement of the old side piers. This is described in the *Engineering Record* for May 31, 1913.

The octagonal pier shaft, 54 feet high and averaging 37 feet in short diameter, consisted of thirty courses of dimension stones from 16 to 24 inches high. It rested about 4 feet below water level on an octagonal grillage, from 44½ to 56½ feet in short diameter, which was composed of seventeen courses of 12×12-inch timbers on a pile foundation. The stone work approximated 2190 cubic yards and weighed about 4732 tons. The grillage contained 78,000 feet board measure of white pine and 355,000 feet board measure of white oak, which together had an average weight of about 69 pounds per cubic foot, computed from the floating displacement. There were 477 foundation piles, all of which had been in place since about 1886.

The stones were wedged apart and removed by a floating derrick, very little drilling and blasting being required, so that most of the material was salvaged for future use. The stones below water level were removed in the dry by the use of the old coffer-dam constructed for the original erection of the pier, and were left in position after the completion of this work. The coffer-dam consisted of five courses of 6×12-inch timber laid edgewise around the periphery of the grillage. This was reinforced and pumped out sufficiently for the requirements of the work.

After the removal of all of the stone work the coffer-dam was again pumped dry and the buoyancy of the grillage, aided by the lifting power of the dredge, enabled the grillage to be raised clear of the foundation piles and floated. It was towed to a convenient point and two pile bents were driven as closely as possible to it, diametrically opposite each other. Three sets of 1½-inch chains suspended from the pile caps were attached to the grillage and were jacked up to raise the latter. As the grillage was lifted the successive courses, most of which were fastened in both directions by 30-inch drift-bolts 3 to 4 feet apart, were wedged apart and pulled off by a floating derrick. This work was accomplished with considerable difficulty at first, but later on it was performed more easily and resulted in salvaging about 91 per cent of all the timber.

After the removal of the grillage from the foundation piles, the mud around the tops of the latter was dredged where necessary to a depth of 5 to 6 feet, by a clam-shell bucket and the old piles were driven down by the use of a heavy timber follower about 30 feet long, with an iron ring at the lower end, which was placed over the heads of the piles by a diver, who followed them down to an average depth of about 5 feet. None of the old foundation piles was pulled or broken off, but they were followed down, to give a clear depth of 25 feet over the top of piles. The old broken protection piles

surrounding the pier were either pulled or cut off square by divers.

The removal of the stone work was commenced Sept. 7 and completed Nov. 29, 1912. The removal of the grillage was commenced Dec. 10, 1912; the separation of its timbers was commenced Jan. 25, 1913, and was finished Feb. 25. The driving down of the old piles was commenced Dec. 16 and finished Feb. 7.

The old Eleventh Street bridge piers at Tacoma, Wn., removed by the author during the construction of a new steel bridge,

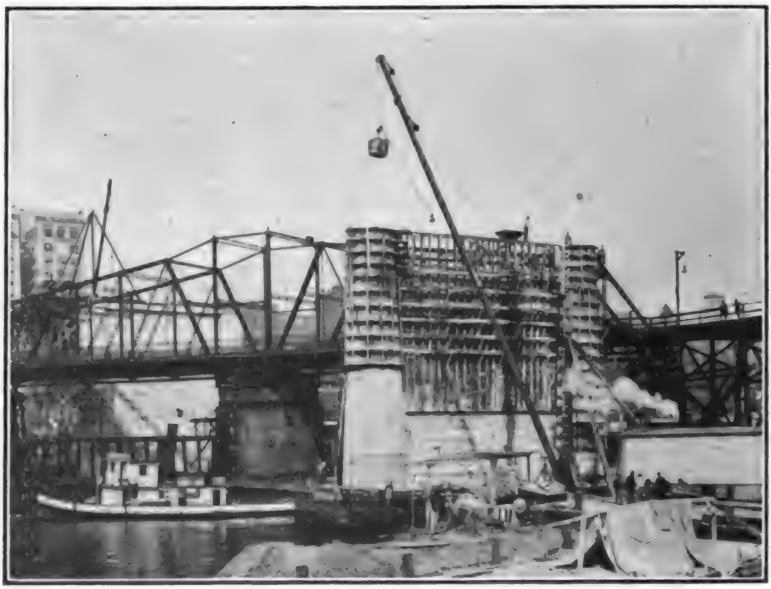


FIG. 105.—OLD PIVOT PIER, TACOMA, WN.

were steel cylinders filled with concrete and resting upon piles driven into the sand bottom. Three of the piers, 44 feet long, were made up of two cylinders, one set 8 feet in diameter and two sets 10 feet in diameter braced together with latticed struts and rods. The old pivot pier (Fig. 105), was 34 feet in diameter and 62 feet total height, the steel shell being filled solid with concrete up to within about 20 feet of the top, above which it consisted simply of a ring of concrete averaging about 5 feet in thickness to carry the circular track, while plate girders crossing at right angles at the top supported the center.

The bracing between the two cylinders forming one pier was removed, and a clam-shell bucket on one of the derrick scows used

to dig out the material around each tube, down to considerably below the bottom of the pier. Heavy lashings of 1-inch cable were placed on each cylinder a short way from the top and bottom, and then the piles holding up the tube shot off by aid of a diver, so that the tube was dropped over on its side. Each one weighed about 150 tons, so that two scows were employed to pick it up and take it out to deep water. Across these two scows were placed two fir saw-logs about 4 feet in diameter, resting on 12×12 timbers on the scows to distribute the load, as shown in Fig. 106, and then the diver picked up the lashings and made them fast around the logs at low tide. With an 18-foot rise in the tide the scows lifted the

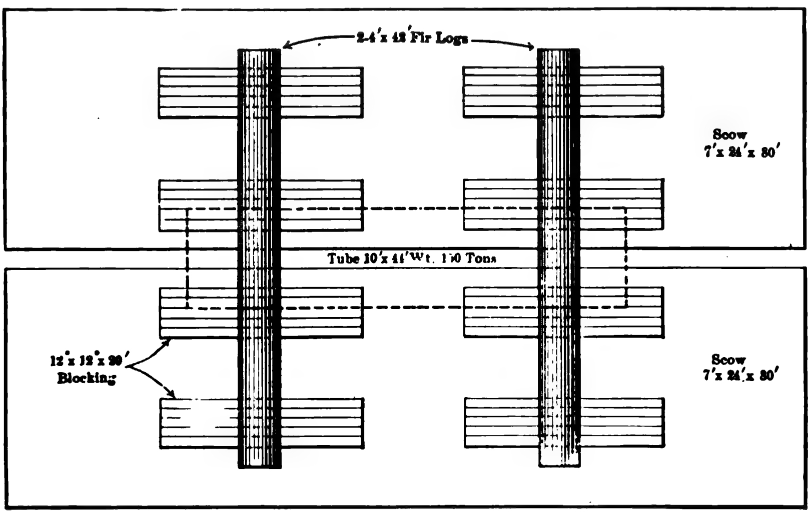


Fig. 106.—Scows for Disposing of Old Piers.

tube clear of the bottom and it was then towed out by the tug to deep water, where the depth was about 150 to 200 feet, at a distance of a mile away from the bridge. The wire-rope lashings were let go and the tube rolled out and dropped to the bottom, the clamps having been previously removed and arranged for this purpose. The same operation was repeated with the five remaining tubes, and the crew became so expert at it that it was only a matter of a few hours to wreck one of the tubes and drop it in deep water.

The removal of the center pier shown in Fig. 107 was a more serious undertaking, and a coffer-dam would have been placed around it except for the limited channel room, consequently it was decided to drill the concrete, and break it up with dynamite so that it could

Fig. 107.—Old Tacoma Pier During Erection of New Bridge.

be removed with skips operated from the derrick scows. As fast as the concrete was shot down the curved plates were cut loose and removed one ring at a time, and when the pier had been demolished and removed down to about 3 feet above low water, the heads of the piles holding up the pier were reached and they were bored with a wood auger to a depth of about 12 feet. The holes were then loaded with from twenty to forty sticks of dynamite and four holes shot off at one time with the battery, breaking out a section of the pier on one side and dropping the broken concrete to the bottom, where it could be dredged up with an orange-peel bucket. Before these shots were fired the curved plates were removed clear to the bottom of the pier. After all of these shots in the piles had been fired, and the material picked up with a dredge, the divers started in and shot off the piles in rows, allowing the remaining concrete, which was very friable, to break to pieces and drop to the bottom, where it could be picked up with the orange-peel. Row after row was shot off in this way, until the entire pier was removed down to the elevation required by the United States Engineers; no requirement having been made as to the removal of the stubs of the piles, which would not interfere with future dredging, but would drop out and pop to the surface as the dredge reached them.

The work occupied about six months, the progress having been very slow on account of the fact that very small charges of dynamite were allowed, ranging from five to forty sticks at a time, although no serious jar was felt on the new bridge adjacent. Some of the concrete dropped to the bottom in chunks too large to be picked up by the dredge, and usually four or five sticks of dynamite laid on them by the diver, and discharged by the battery, broke them up in small enough pieces to be dredged out without any trouble.

The old bridge of the Oregon-Washington Railway and Navigation Company at Portland, Ore., was built about the year 1889 from the plans of Geo. S. Morison, Consulting Engineer, and was replaced in 1912, thus having lasted about twenty-three years under very severe usage and a very great increase in the train and street loading. The lower deck carried a single-track railway, while the upper deck carried the street traffic and electric double-track lines on the roadway and foot-passenger traffic on the two narrow sidewalks.

The bridge consisted (Figs. 108 and 109) of a 340-foot draw span, a 320-foot fixed span on the east side and several shorter spans on the west shore. The pivot pier was a steel cylinder 31 feet in diameter filled with concrete and resting on piles and a grillage, the piles having been cut off below low water in the usual way. The

other principal pier, at the east end of the draw span, was of cylinders 14 feet in diameter, filled with concrete and resting on piles, nineteen to each cylinder and extending up into the concrete. The main pier on the west bank was of smaller tubes but constructed in the same manner, and there were three piers still smaller, on shore at the west side of the river.

The bridge spans the Willamette River in the City of Portland, eight miles above its confluence with the Columbia. The highest water usually occurs in June and is due to high water in the Columbia. The extreme range is 28 feet. A flood frequently occurs in winter or spring from the rising of the Willamette and these floods have been known to rise 20 feet above low water, but this is an unusual

FIG. 108.—PIERS OF OLD STEEL BRIDGE, PORTLAND.

occurrence. These Willamette floods are of course accompanied by a considerable current, but during the highest stage, due to the back-water from the Columbia, there is either no current at all or else a slight current up-stream.

No drift runs except during the flood from the Willamette. These considerations making a very short season for work, the closeness of the bridge to the new structure and to structures on the bank, as well as the very frequent passing of boats, rendered the problem of removing the old piers a difficult one.

The draw span was swung around over the draw rest (Fig. 108), and blocked up while being dismantled and taken apart. The 320-foot fixed span was falseworked and removed with very little difficulty. Then the draw rest was pulled apart and the piles broken off or pulled.

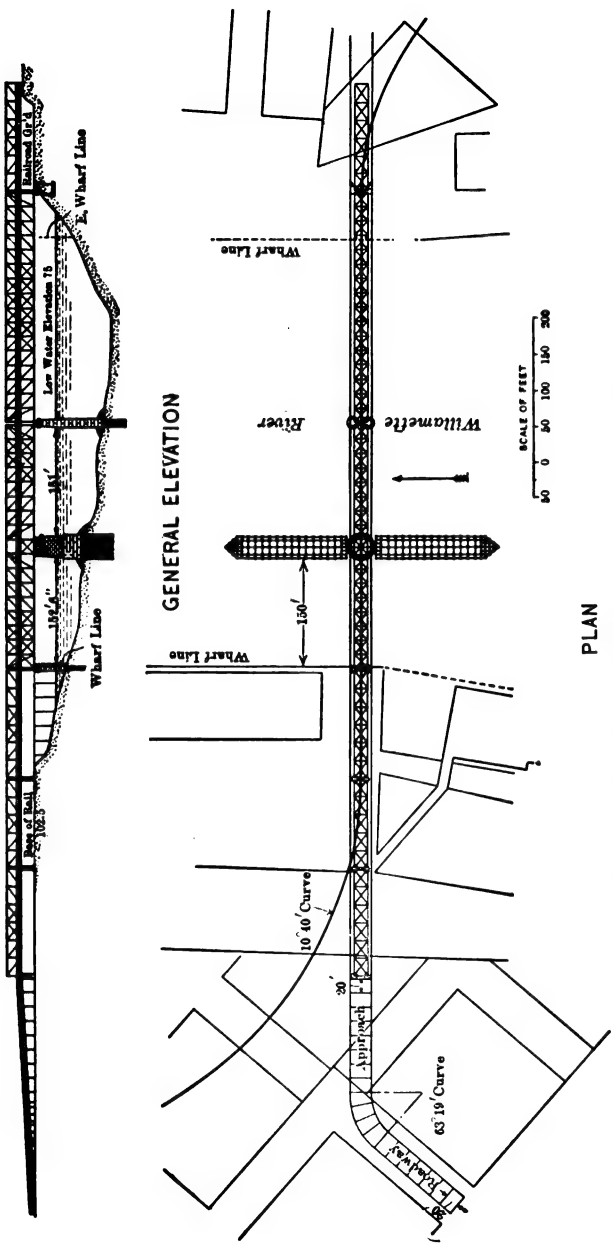


FIG. 109.—PROFILE OF STEEL BRIDGE PIERS, PORTLAND.

Owing to the fact that much of the channel had a depth (Fig. 194) of from 40 to 60 feet at low water, the Government Engineer Department allowed the piers to be blasted apart and shot off down to the depth required for navigation at low water, and the piers or material composing them deposited in the deepest places.

No particular trouble was experienced in the cutting apart of the metal work above the water, nor in removing the concrete down to that point. Owing to the very considerable depth, coffer-dams were out of the question, and it became necessary to either shoot the piers apart by the aid of a diver, and then dredge the material out into or onto scows, and dump it in deep water; or else dredge around the piers, shoot off the supporting piles, tip the tubes over into the deep water or in case they did not land in deep enough water, have the diver attach wire rope tackle to them, and drag or roll them into deep water by the aid of tugboats.

The undermining of the two cylinders composing the east pier was accomplished without any particular trouble, and by the aid of a diver the piles were shot off, so that the pier tipped over into deep water where it required no further attention. The center pier was undermined and the grillage pulled apart so that it tipped over part way into deep water, but still stood up at an angle which brought part of it above the depth required for navigation. This portion of it was shot off by a diver using charges as high as 200 pounds at one time, causing severe jars to surrounding buildings and structures. A very considerable time and about $1500 worth of dynamite were expended before it was removed to the required depth.

The pier at the west bank was allowed to remain for service as part of a wharf foundation, but the plates were cut off of the ones on shore, the concrete broken up and all hauled away by cars.

The foregoing accounts indicate that where the workmanship on piers has been first class, it costs more for labor in many cases to remove them than to construct them in the first instance. For this reason, if no other, it is incumbent upon the engineer to look ahead and so carefully plan, if possible, that the piers at least can be utilized in supporting a new superstructure when it is required.

CHAPTER IX

PUMPING AND DREDGING *

THE degree of success which has been attained in the building of a coffer-dam will be evident when the pumping process is begun. After having been pumped out, if the leakage is so small as to require only a small amount of pumping to keep it free from water, it may reasonably be considered a success.

The pumping should not exceed what can be done by a steam-siphon, a small pulsometer, or by running a centrifugal pump intermittently. Should leaks develop which cannot readily be contended with, then repairs must be made.

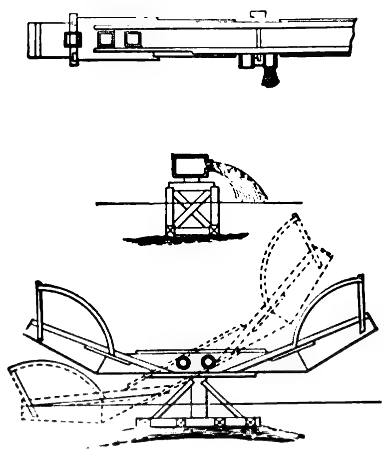


FIG. 110.—OLD BASCULE PUMP.

The use of pumps for this class of work on ancient bridges is described by Cresy. The bascule, used by Perronet at the bridge of Orleans (Fig. 110), is one of the most primitive forms. It consists of a seesaw apparatus, at each end of which ten men were placed, and 150 motions were given in it each quarter of an hour. Four cubic feet of water were raised 3 feet each time, or about 300 gallons per minute. Various other kinds of pumps were used at this bridge, among them the chaplet, which is similar to a modern chain-pump, worked by hand. Then the same device was employed, but geared to be operated by horses on a platform. A chaplet operated by a

* Attention is called to the numerous references in other chapters to the pumping plants actually employed on coffer-dams, and especially to the plant used at Topeka.

Great care should always be given to the selection of a pumping plant of the proper type and proper size, as the statements regarding capacity are often misleading. The outfit should be, if needed, one able to take care of the dredging, if the material is such that it can be pumped.

water-wheel was also used. (Figs. 111 and 112.) The large wheel had 124 cogs, while the pinion had 15, which caused the raising of over sixty-six buckets on the chain for each turn of the large wheel. At 180 turns of the wheel per hour, with each bucket lifting 290 cubic inches of water, the capacity was about 250 gallons per minute.

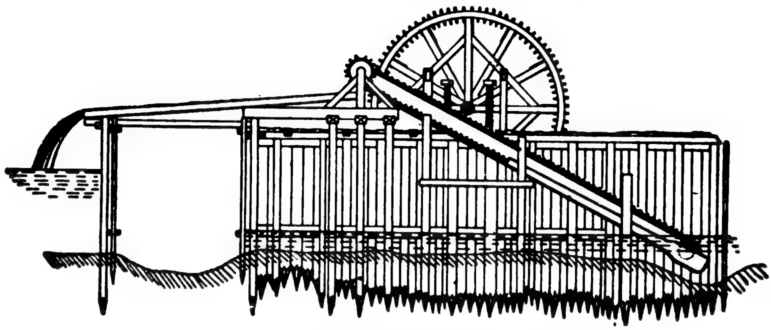


FIG. 111.—OLD CHAPELET, SIDE ELEVATION.

A great bucket-wheel was employed by the same engineer at the Neuilly bridge, 16 feet 6 inches in diameter, 4 feet 6 inches wide, with sixteen buckets.

The pumps used at the present time on very small work are usually square wooden-box lift-pumps, such as are used on large river barges, and are worked by one or more men lifting on a plunger.

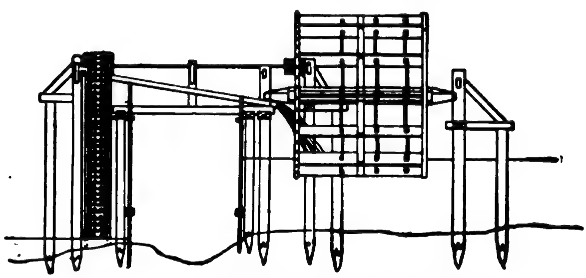


FIG. 112.—OLD CHAPELET, END ELEVATION.

These are often replaced by a similar pump of metal (Figs. 113 and 114) with a tube of galvanized metal, and often spiral-riveted. The one shown in Fig. 113 has the top and bottom soldered to the tube, while the one in Fig. 114 has screw joints. The cost of a 4-inch pump 8 feet long with fixed top and bottom would be about $9, while the screw joints would about double the cost.

Such pumps are, however, little used, as the labor becomes excessive where there is any quantity of water to deal with, and

diaphragm pumps (Fig. 115) are employed, which work on a rubber diaphragm in place of a piston and plunger, and throw a large amount of water, besides allowing the passage of sand and gravel without choking the pump. The 2½-inch suction has a capacity of 25 gallons per minute, and the 3-inch suction of 58 gallons per minute, the list price of the two sizes being $40 and $62, respectively; the maximum lift of the pump being 30 feet.

Diaphragm pumps already described that are operated by hand are rapidly being superseded by diaphragm pumps operated by gasoline engine (Fig. 116), and they are, of course, very much more

Fig. 113.
Hand-pump,
Soldered Joints.

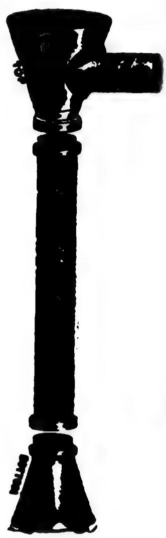


Fig. 114.
Hand-pump,
Screw Joints.

Fig. 115.
Diaphragm Pump.

efficient and economical than similar pumps operated by hand. They should be used only where judgment dictates that they are cheaper than some type of steam pump, or else it is impossible to use a steam-driven pump, on account of the expense of moving the steam plant. The No. 3 pump, with 3-inch suction, weighs 750 pounds and has a capacity of 3000 gallons per hour, while the No. 4, with a 4-inch suction, weighs 790 pounds and has a capacity of 6000 gallons per hour. Both are operated by 3-horse-power gasoline engines.

Where steam can be obtained steam-siphons are often used, the steam being introduced into the main pipe through a nozzle, thus causing a suction, which with a 3-inch discharge Van Duzen

jet will deliver 7200 gallons of water per hour, the height of the pump above water being 11 feet, the point of discharge being 19 feet above the pump, making a total lift of 30 feet. This size will require an 18-horse-power boiler and a steam pressure of 50 pounds. The suction-pipe is 1 inch larger than the discharge, while the steam-pipe is $1\frac{1}{4}$ inches in diameter, with a jet opening of about $\frac{13}{16}$ inch.

The list price of a pump of this size (Fig. 117) is $75, the piping being extra. The pump is constructed of gun-metal and will last indefinitely. The strainer should always be used and will cost about $8 extra for the 4-inch pipe. The piping should have long bends in place of elbows where a turn is required.

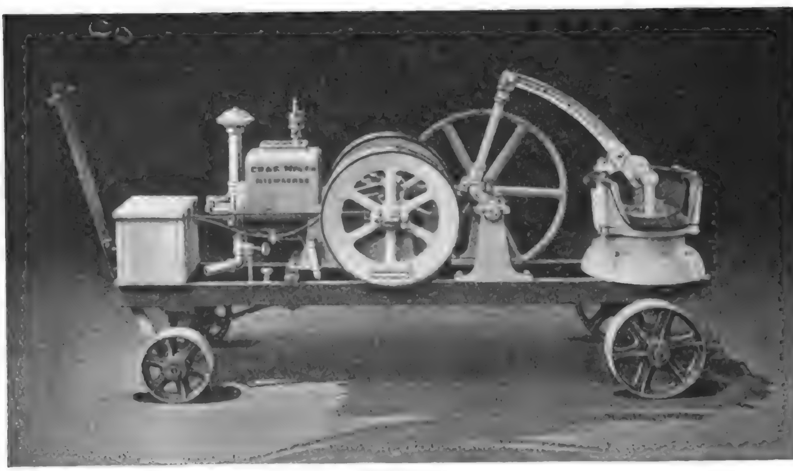


FIG. 116.—GASOLINE DIAPHRAGM PUMP.

This make of pump is manufactured from $\frac{1}{2}$-inch discharge, with a capacity of 200 gallons per hour, up to 5-inch discharge with a capacity of 12,000 gallons per hour. The smaller sizes are useful for priming centrifugal pumps and for a variety of uses around a contractor's plant.

The Lansdell siphon-pump (Fig. 118) has a double suction *CC*, to which rubber suction-pipes are attached. The steam-pipe is attached to *B*, and when the steam is turned on it is blown across *A* and through *D*, thus exhausting the air from the chamber *A*. Water rises through *CC* by atmospheric pressure to fill the vacuum, and it is forced out through *D* by the steam, the velocity being proportional to the steam pressure. The steam supply should be as close to the pump as possible, to prevent condensation, and the

turns in the pipe should be easy bends, as stated regarding the Van Duzen jet. When the height to which the water is to be pumped exceeds 14 feet, the suction-pipes must be long enough to allow the center of the pump to be placed 14 feet above the water. With a 3-inch discharge, a 1½-inch steam-pipe is required and a 12-horse-power boiler. With a 6-inch discharge a 2½-inch steam-pipe is required and a 50-horse-power boiler.

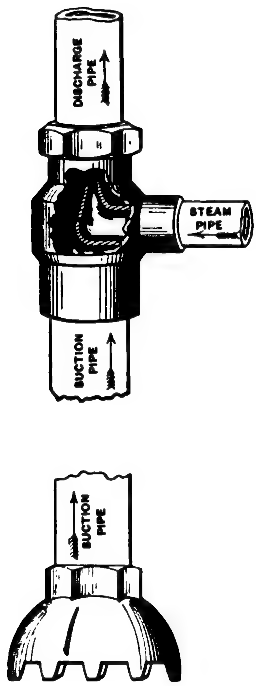


FIG. 117.—VAN DUZEN JET-PUMP.

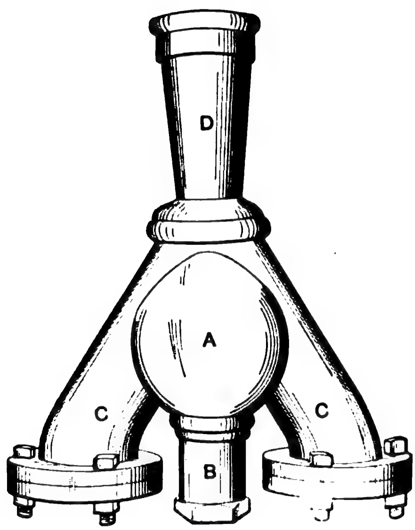


FIG. 118.—LANSDELL'S SIPHON-PUMP.

The rated capacity of the 3-inch is 450 gallons per minute; of the 6-inch, 1800 gallons. But this would likely not be realized in practice.

The vacuum-pump which has reached the most general adoption is the pulsometer, and is in many ways better adapted to light service than a centrifugal pump of small size. There are no bearings to keep up, no belts to keep tight, and no trouble in preparing a foundation, as the pump is suspended by the hook shown in Fig. 119. The pump is operated by admitting the steam through the pipe at the

extreme top (Fig. 120), the pump having been previously primed by filling the middle chamber with water. The air-valves are closed and the steam passes into the right-hand chamber *A*, clearing it of water by forcing it into the discharge-chamber shown in dotted lines. The steam then condenses at once and the ball *C* changes its seat,

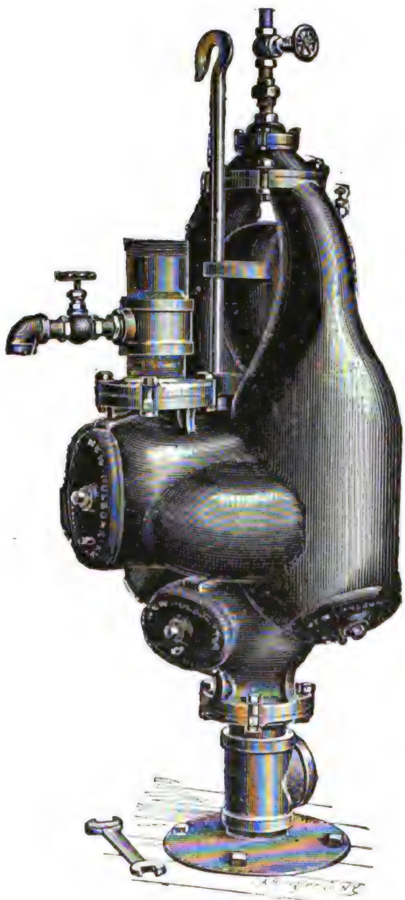


FIG. 119.—PULSOMETER STEAM-PUMP.

FIG. 120.—SECTION OF PULSOMETER.

closing the right-hand and opening the left-hand chamber to the steam. The vacuum, formed by the steam condensing in the right-hand chamber *A*, allows it to fill with water by atmospheric pressure through the suction pipe at the extreme bottom and through the chamber *D*, it being retained by the valves *E, E*. The steam then

enters the left-hand chamber *A* and the operation is repeated. The chamber *J* is a vacuum-chamber.

In starting the pump the steam is turned on for three or four seconds, then shut off for four or five seconds, alternating these movements until the pump is started. The steam is then turned on about half or three-quarters of a revolution, the two side air-valves opened about half a turn, and then the middle air-valve opened slowly until a regular stroke is obtained.

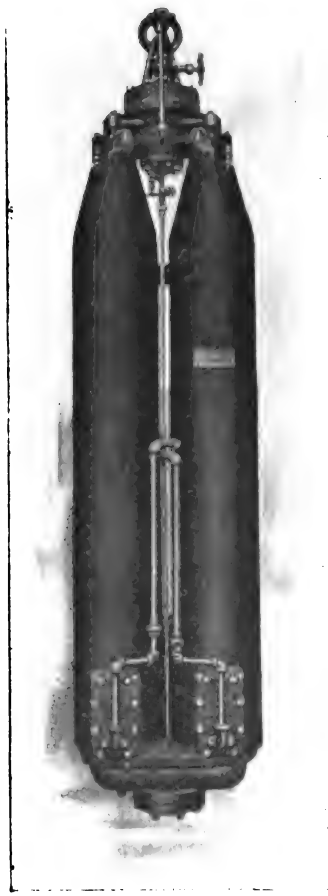


FIG. 121.—EMERSON PUMP.

The capacity of the 3-inch discharge, with a $\frac{1}{2}$-inch steam-pipe and operated by a 9-horse-power boiler, is 180 gallons per minute when the lift is as much as 25 feet; and for the 6-inch discharge, with a $1\frac{1}{2}$-inch steam-pipe and operated by a 35-horse-power boiler, 1000 gallons for the same lift.

The pulsometer is remarkably smooth in operation, and except for the slight click of the ball and the discharge of water in a steady stream, one would scarcely know it was pumping. Where a good-sized hoisting-engine boiler is in use on foundation work, it can be used to supply the steam for pumping. The work illustrated in Fig. 4 was easily kept free of water by a small pulsometer, while its use has been cited in a number of cases where the cofferdam was pumped out by a centrifugal pump, and then the leakage kept under control by a medium-sized pulsometer, which required but little attention. The pump should be provided with a strainer at the bottom of the suction-pipe, all the connections must be air-tight, no sharp bends should be made in the pipe, and with dry steam successful working will result. Another pump of similar construction is the Maslin automatic vacuum-pump, which differs from it

in important details. What has been said regarding the pulsometer will apply as well to the Maslin pump.

The Emerson foundation pump is one which has been much used of late years and gives much better service in pumping out coffer-dams or cribs, than any other style of pump except the centrifugal. This pump is shown in Fig. 121 in elevation, and in section in Fig. 122. The sizes of these pumps are given in Table XIV. In using them great care must be taken to see that the piping on the pump is properly connected up as originally received from the factory, or as may be learned from the instructions accompanying each pump. They can be swung from a derrick or hung up by sling around a timber in a coffer-dam, and as they take up such a small amount of space horizontally, they will be found very convenient, especially where the working room is limited.

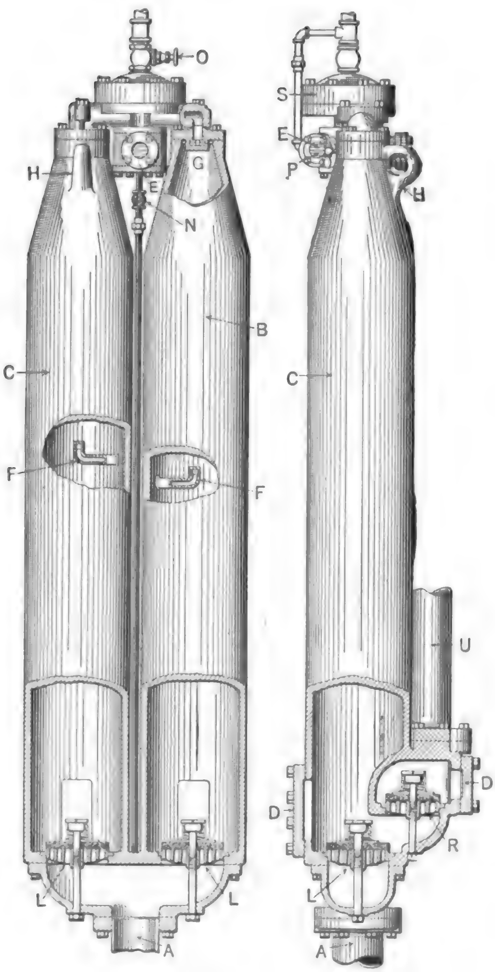


Fig. 122.—Emerson Pump, Section.

All the foregoing devices are for use where the amount of water to be handled in a given time is of limited amount, but where large quantities are to be pumped out of coffer-dams in short periods of time, recourse must be had to centrifugal pumps, which have reached a high state of perfection. Where the water is to be lifted 10 feet an ordinary reciprocating pump would exhibit an efficiency of only 30 per cent., while a centrifugal pump would have an efficiency of

TABLE XIV.—SHOWING SIZES, CAPACITIES, WEIGHTS, PRICE LIST AND OTHER DATA OF EMERSON STANDARD STEAM PUMPS

Cipher Code Word.	Number.	Diameter of Cylinders. Inches.	Length of Cylinders. Feet.	Size of Steam Pipe. Inches.	Size of Suction. Inches.	Size of Discharge. Inches.	Capacity in Gallons per Minute.	Capacity in Gallons per Hour.	Capacity in Gallons per Day, 24 Hrs.	Dimensions over all in Inches.			Approximate Weights in Lbs.	Price List.
										Breadth.	Width.	H.		
Aid............	1	6	6	¾	3	2½	225	13,500	324,000	16½	18	97½	950	$275
Ark............	2	8	6½	1	4	3	415	24,900	597,600	21½	21	104	1375	350
Alps...........	3	10	7	1¼	5	4	725	43,500	1,044,000	26	24	113	1900	500
Agate..........	4	12	8	1½	6	5	1200	72,000	1,728,000	29½	27¼	127	3100	700
Adam..........	5	16	8	2	8	6	2100	126,000	3,029,000	43¼	33	132	4400	1,150
Amos..........	6	20	8	2½	10	8	3275	196,500	4,716,000	51½	36¾	135	5400	1,700

Capacities stated in table, in gallons per minute and per hour, are calculated on a head or lift of 20 feet. These diminish at the rate of about 4% for every 10 feet additional head up to 150 feet, the highest head for which we recommend our Standard Pumps.

64 per cent. For a lift of 17 feet the reciprocating type would have an efficiency of 50 per cent., while the centrifugal would reach its maximum of 69 per cent. efficiency, dropping to only 50 per cent. for a lift of 50 feet, while the other types would increase to 75 per cent. From this it will be seen that the centrifugal pump is essentially a low-lift machine.

Actual tests of pumps show that the maximum results are very seldom realized, a 9-inch discharge of one make showing an increase from 46.52 per cent. for a 12.25-foot lift to 57.57 per cent. for a 13.08-foot lift, while another make of 10-inch discharge shows a decrease from 64.5 per cent. for a 12.33-foot lift to 55.72 per cent. for a 13-foot lift. The greatest efficiency at hand is shown by a German pump with a $9\frac{1}{4}$-inch discharge, a 10.3-inch suction and a 20.5-inch disk, running at 500 revolutions. The lift was 16.46 feet and the efficiency 73.1 per cent.!

That such results are not realized on actual work is readily understood when it is considered what little care is used to properly place and operate such a plant, how little attention is paid to having a proper boiler and engine, and what lack of care there often is to keep the plant in good repair.

An ideal outfit for operating by steam is shown in Fig. 123, where the engine is directly connected to a Heald & Sisco pump, all the trouble and vexation from the use of a belt being done away with, and no loss of power through slipping of belts. The machine can be placed on the barge which carries the boiler, the suction-pipe being run horizontally across as in Fig. 123, while a short discharge-pipe discharges directly into the river. Where electric power plants are available a still better arrangement will be to have an electric motor directly connected to the pump, and all the trouble incident to the use of a boiler on the work will be avoided.

Electric power can also be used for hoisting and for pile-driving. Examples of the use of motors on hoisting machinery will be given in a later article.

The suction should always be fitted with a section of smooth-bore rubber hose (Fig. 124, *a*) to give it flexibility, a length of about 8 feet being usually sufficient. The best hose is made with a spiral metal core, which adds to its strength and durability.

The suction-pipe is ordinarily made of sections of wrought-iron pipe, with screw connections, but as this is troublesome to change sections, it will be found advantageous to use the spiral-riveted pipe with flange couplings (Fig. 124, *b*), and to have extra sections from 2 to 6 feet long, with several sections of each shorter length so that

the length of the suction-pipe can be readily changed to suit the depth of the excavation. The flanges must be provided with rubber gaskets to keep the pipe air-tight.

The strainer (Fig. 124, *c*) is used to prevent large stones, sticks, or obstructions from entering and clogging ordinary pumps, and usually comprises a foot-valve to retain a pipe full of water and make the priming easy. The strainer or end of the suction-pipe is usually

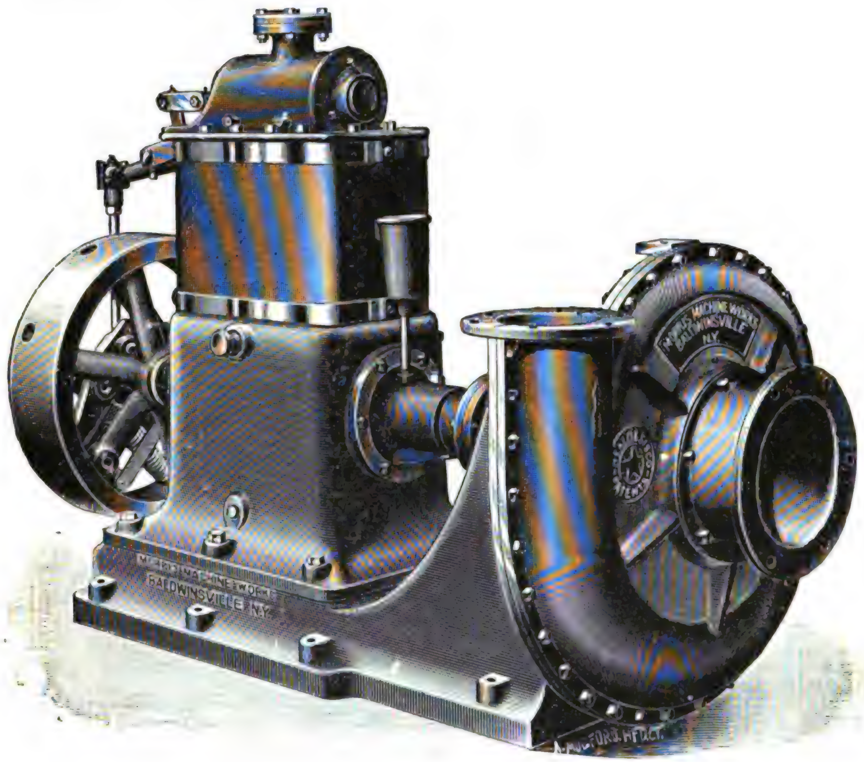


FIG. 123.—CENTRIFUGAL PUMP, DIRECTLY CONNECTED TO ENGINE.

placed in the lowest point, and sometimes a box or sump is provided, as a well into which the water is drained from the other and higher portions of the work. A small set of falls should be attached to the foot to raise the pipe and clean out the strainer when necessary.

The centrifugal pump itself must be in first-class repair to do economical work, and should be a large enough size so that it need not be run beyond its economical capacity. The style of pump to use

will depend upon the work to be done, and for coffer-dam work a vertical pump is not often used, but data regarding it is given. Where practically clean water is to be pumped an ordinary style of pump should be used, but where much mud or sand will be drawn up a sand-pump is best; and where a large part of the excavation is to be done with the pump, as at Topeka, a dredging-pump will be the proper type.

The pumping required on the Chattanooga work, 5000 gallons per minute to a height of about 15 feet, would have been done most economically by a 15-inch pump, with a 40-horse-power engine and a 50-horse-power boiler. But a pump of this size would not find ready use in a contractor's work, and for this reason two 8-inch

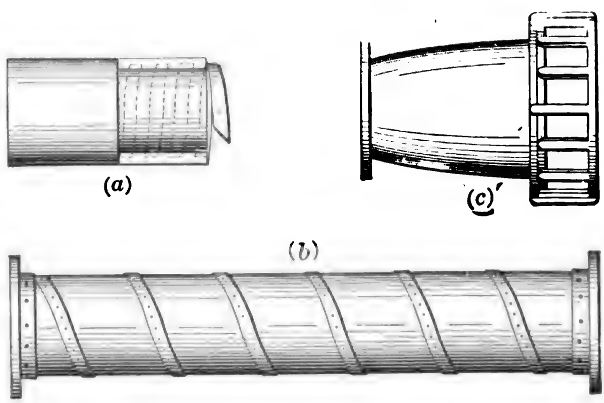


FIG. 124.—SUCTION DETAILS FOR PUMPS.

pumps would have been the better outfit to purchase, unless the work was very extensive; and each pump should be provided with a 25- or 30-horse-power engine, so as to run the pumps somewhat beyond the economical capacity, which could readily be done with a direct-connected engine, where there would be no belt to slip.

The work required on the Forth Bridge coffer-dams could also be done by the 15-inch pump above described, the lift being about 3 feet at the start and reaching 18 feet as the dam was cleared, the 340,000 gallons being pumped out in about one hour.

Centrifugal pumps are rarely required for a lift of over 20 feet on this class of work, which is only slightly beyond the economical lift, and the height should never exceed 30 feet, which would require for the 15-inch pump an engine of 75 horse-power.

The pump may be located on the coffer-dam, but in case of high

water during the progress of the work the outfit may be damaged and it is best to place the pump on a boat, as in Fig. 98, with a section of horizontal suction-pipe across to the work, which should be as short as possible.

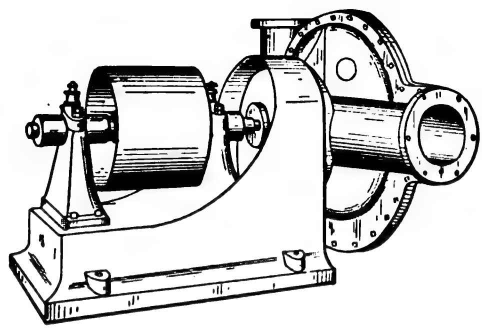


FIG. 125.—CENTRIFUGAL PUMP, DOUBLE SUCTION.

The ordinary type of pump (Fig. 123) may be fitted with a primer, consisting of a small hand force-pump attached to one side of the pump, for filling the pump and suction-pipe. A more simple way is to provide a barrel above the pump, which can be kept full by using a small steam-jet, and, by means of a pipe with valve from the bottom of the barrel to the top of pump, the contents can be emptied into the pump to

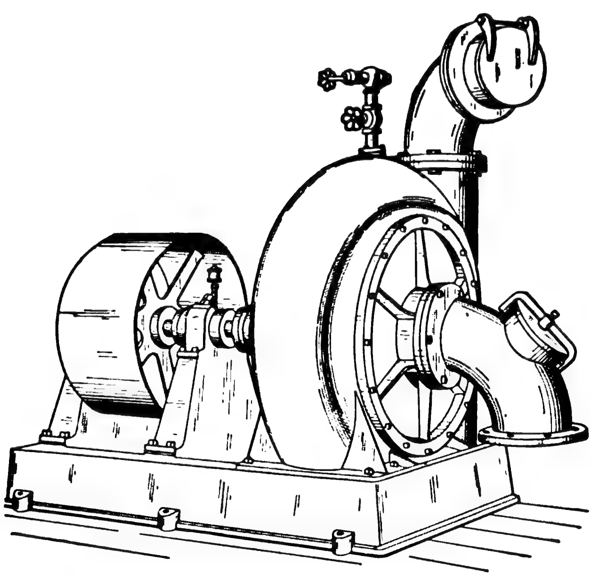


FIG. 126.—DREDGING-PUMP.

prime it. Priming may also be easily accomplished by inserting a hose into the discharge-pipe and filling the pump directly with a steam-jet.

Double suction-pumps (Fig. 125) allow the water to enter on each side of the piston, and thus a perfect balance is secured, which does away with all end-thrust on the bearings. This pump is most easily primed by using an ejector, or a flap-valve such as is shown on the discharge-pipe of the dredging-pump (Fig. 126) and which serves to retain the water in the pump. Where a long discharge-pipe is to be used, a quick-closing gate-valve may be introduced into the pipe near the pump.

Where the material to be dredged out at the foundation site is mud or sand or partly gravel, it can be removed during the process of pumping by using a dredging-pump. In case there were 700 yards of material to be removed and an 8-inch pump was provided, it would not be advisable to count on more than 10 per cent of solid matter being discharged by the pump, as the suction could not be kept working close up to the sand or mud. By using a 30-horse-power engine, a discharge of 2000 gallons per minute would be reached, or with 10 per cent. of loose solid matter the excavation would be made in less than two working days.

FIG. 127.—DREDGING-PUMP PISTON.

The piston of a dredging-pump (Fig. 127) is provided with large openings to receive the material, and the one illustrated is provided with side plates so that all wear is taken off the pump-casing.

The vertical centrifugal pump, Figs. 128 and 129, is one which can be used to advantage in a great deal of coffer-dam work where it is necessary for the pump to be submerged part or all of the time. The pump can be mounted on a timber frame to be raised and lowered with a derrick. Such a pump requires no priming, and is always ready to run as soon as the engine is started. The engine or motor should be some type to be directly connected or geared on to the shaft, or else a belt from a separate engine could be used. It will very often clear a coffer-dam of water where an ordinary pump having long and possibly leaky suction will not work at all. The sizes and capacities of centrifugal pumps are given in Tables XV,

XVI, and XVII, and the number of revolutions to run them to raise water to different heights is given in Table XVIII.

The ejector as used on a centrifugal pump is best for priming; and with a foot valve on the suction, nothing is required except to turn on the steam to the ejector, and operate it until the pump and pipes are filled with water, after which when started, the pump will pick up its prime without any trouble.

One of the most remarkable pieces of work done with this class of pumps was the use of Edwards' cataract pumps in dredging the

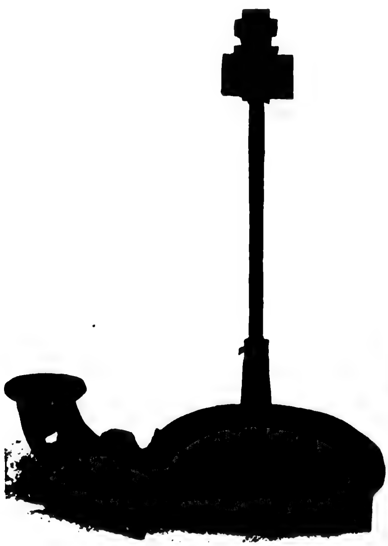


Fig. 128.—Vertical Pump. Submerged Type.

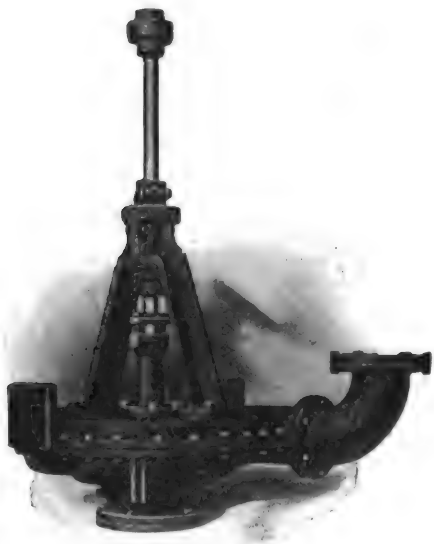


Fig. 129.—Vertical Pump. Suction Type.

ship channel in New York Harbor. This is described in the Trans. Am. Soc. C.E., Vol. 25. The work was done by three dredges, which were much the same as small sea-going vessels, the largest being the *Reliance*, 157 feet long, and carrying 650 cubic yards of dredged material. Two separate pumps were provided, each with 18-inch suction-pipes reaching from the sides of the vessel and parallel to it down to the bottom to be dredged, being supported by suitable hoisting-tackle. These boats were kept under headway toward the dumping-ground while the dredging was in process. The average load during about a month's working of the *Reliance*

TABLE XV.—HEALD & SISCO STANDARD IRON HORIZONTAL CENTRIFUGAL PUMPS

No.	Capacity in Gallons per Minute.	Horse-power Required for each Foot of Lift. Min. Quantity.	Diameter and Face of Pulley in Ins.	Floor Space Required in Inches.	Shipping Weight, Pounds.	Price of Pump, Oilers and Wrench.	Price of Pump and Primer.	No.
1¼	50 to 70	.024	6×6	17×30	168	$45	$55	1¼
1½	75 to 100	.037	7×8	21×33	232	60	70	1½
2	110 to 150	.054	8×8	23×37	306	75	90	2
2½	175 to 250	.086	8×8	24×38	348	90	105	2½
3	250 to 350	.124	8×8	25×39	400	110	130	3
4	450 to 600	.223	10×10	30×40	545	130	155	4
5	750 to 900	.372	15×12	34×54	826	165	195	5
6	1,000 to 1,400	.496	15×12	37×55	965	200	240	6
8	1,700 to 2,200	.844	20×12	45×63	1,500	310	375	8
10	2,200 to 3,000	1.093	24×12	51×71	2,170	395	470	10
12	3,000 to 4,000	1.49	30×14	62×78	3,050	500	...	12
15	4,800 to 6,000	2.38	40×15	77×80	7,100	850	...	15
15*	4,800 to 6,000	2.38	30×15	60×68	3,150	710	...	15
18	7,500 to 10,000	3.73	40×15	93×103	9,000	1,300	...	18
18*	7,500 to 10,000	3.73	30×16	62×70	3,500	1,150	...	18
22	12,000 to 14,000	5.96	48×20	126×130	12,000	......	...	22

* Refers to low-lift pumps.

The number of pump is also diameter of discharge opening in inches. Where more than 25 feet of discharge-pipe is attached to pump, use one or two sizes larger than pump-discharge.

For No. 12 and larger sizes a foot-valve or flap-valve and ejector for priming is recommended.

TABLE XVI.—LIST OF HEALD & SISCO HYDRAULIC DREDGING AND SAND-PUMPS

Number of Pump.	Diameter Suction and Discharge Openings, Inches.	Cubic Yards of Material they will Raise per Hour.	Horse-power Recommended for 10-foot Lift.	Diameter and Face of Pulley.	Floor Space Required, Inches.	Shipping Weight, Pounds.	Will Pass Solids, Diameter, Ins.	Price of Pump Complete, with Suction and Discharge Elbows, Flap-valve, and Ejector.	Number of Pump.
4	4	30 to 50	6	12×12	40×31	800	2	$210	4
6	6	60 to 80	12	20×12	68×40	1,700	4½	300	6
8	8	125 to 150	22	24×14	72×48	3,400	6	475	8
10	10	200 to 300	35	30×15	94×54	4,200	8	600	10
12	12	300 to 375	45	36×20	114×66	9,000	10	850	12
15	15	400 to 500	75	42×24	154×78	12,000	10	1,450	15
18	18	500 to 700	125	48×30	160×80	13,500	10	1,900	18
20	20	....	.....	......	.....	.....	....	....	20
22	22	....	.....	......	.....	.....	....	....	22

TABLE XVII.—VERTICAL CENTRIFUGAL PUMPS

No. Pump (Diameter Discharge Opening)	Economical Capacity in Gallons per Minute.	Horse-Power Required for Each Foot Elevation.	Diameter and Face of Pulley in Inches.	Floor Space Required in Inches.	Distance Bottom of Pump to Bottom of Center of Coupling.	Coupling Bored for Connecting Shaft, Inches.	Price Extra Bearings, Each.	Price Extra Couplings Each.	Shipping Weight Pounds, Submerged Type.	Price Complete as per Foot Note, Submerged Type.	Shipping Weight, Pounds, Suction Type.	Price Complete as per Foot Note Suction Type.	No. Pump.
$1\frac{1}{2}$	70	.058	5×6	17×21	2 ft. 9 in.	1	$1.00	$1.50	110	$40	135	$62	$1\frac{1}{2}$
$1\frac{3}{4}$	90	.075	6×6	21×29	3 " 0 "	1	1.00	1.50	165	50	200	78	$1\frac{3}{4}$
2	120	.10	7×8	23×30	3 " 4 "	$1\frac{1}{16}$	1.50	2.00	198	65	250	100	2
$2\frac{1}{2}$	180	.15	7×8	24×30	3 " 4 "	$1\frac{1}{16}$	1.50	2.00	220	80	275	124	$2\frac{1}{2}$
3	260	.22	7×8	25×32	3 " 6 "	$1\frac{1}{4}$	1.50	2.00	235	95	340	147	3
4	470	.30	8×10	29×39	4 " 0 "	$1\frac{7}{16}$	2.00	2.50	380	110	495	170	4
5	735	.45	10×10	34×45	4 " 7 "	$1\frac{11}{16}$	2.50	3.00	605	140	785	216	5
6	1,050	.59	12×12	37×48	4 " 7 "	$1\frac{13}{16}$	3.00	3.50	740	170	1,050	285	6
8	2,000	1.00	18×12	45×56	5 " 5 "	2	4.00	4.00	1,320	265	1,710	445	8
10	3,000	1.52	20×12	51×68	5 " 5 "	2	4.00	4.00	1,430	330	1,925	550	10
12	4,200	2.00	24×14	63×72	6 " 0 "	$2\frac{3}{8}$	5.00	5.50	2,640	420	3,000	700	12
12*	4,200	2.00	20×12	49×62	3 " 9 "	$2\frac{5}{8}$	5.00	5.50	2,000	370	2,500	650	12*
15	7,000	3.50	30×16	77×102	6 " 6 "	$3\frac{1}{4}$	8.00	8.00	5,500	600	6,000	1,000	15
15*	7,000	3.50	30×15	60×71	6 " 6 "	$3\frac{1}{4}$	8.00	8.00	2,650	480	3,100	800	15*
18	10,000	4.50	36×18	98×126	7 " 0 "	$3\frac{3}{4}$	10.00	12.00	6,000	950	7,000	1,585	18
18*	10,000	4.50	30×16	66×78	6 " 6 "	$3\frac{1}{4}$	10.00	12.00	2,900	850	3,300	1,420	18*
20	12,000	5.40	36×20	73×92	4 " 6 "	4	15.00	20.00	4,500	1,255	5,200	2,100	20
24*	15,000	6.50	48×20	88×110	6 " 0 "	$4\frac{1}{2}$	20.00	25.00	8,000	2,000	9,000	2,800	24*

* Refers to low-lift pumps.

was 585 cubic yards and the average time of loading about forty-eight minutes, while the average number of loads per day was 6.73.

TABLE XVIII.—NUMBER OF REVOLUTIONS AT WHICH PUMPS SHOULD RUN TO RAISE WATER TO DIFFERENT HEIGHTS

No.	5 Feet.	10 Feet.	15 Feet.	20 Feet.	25 Feet.	30 Feet.	35 Feet.	40 Feet.
1¼	428	604	739	854	955	1045	1131	1208
1½	348	491	601	695	777	850	920	982
2	302	426	522	603	674	737	798	852
2½	302	426	522	603	674	737	798	852
3	302	426	522	603	674	737	798	852
4	285	402	493	569	637	697	754	805
5	256	362	443	512	572	626	678	724
6	214	302	368	427	478	523	566	604
8	183	259	317	366	409	448	485	517
10	168	238	291	336	376	411	445	475
12	133	188	230	266	298	326	352	376
15	105	148	181	209	234	256	277	295
15*	151	213	261	301	337	369	399	426
18	105	148	181	209	234	256	277	295
18*	151	213	261	301	337	369	399	426

Above table gives *correct* speed of pumps as employed under usual conditions of pumping. If water must be forced through a number of bends and elbows, or a great length of piping, the above speed must be somewhat increased.

Use large pipes and easy bends wherever practicable, as they save power.

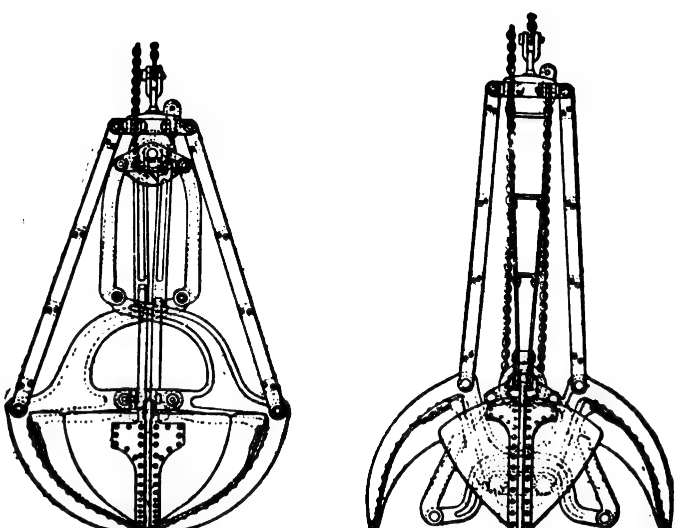


FIG. 130.—LANCASTER GRAPPLE.

These dredges removed the enormous quantity of 4,299,858 cubic yards of material at an average price of 24.48 cents per yard, the lowest price being about 17 cents, the average price paid for other forms of dredging being 40.53 cents. On foundation work the amounts to be removed would be small and the cost for this reason much higher, yet owing to the smaller cost of the plant that

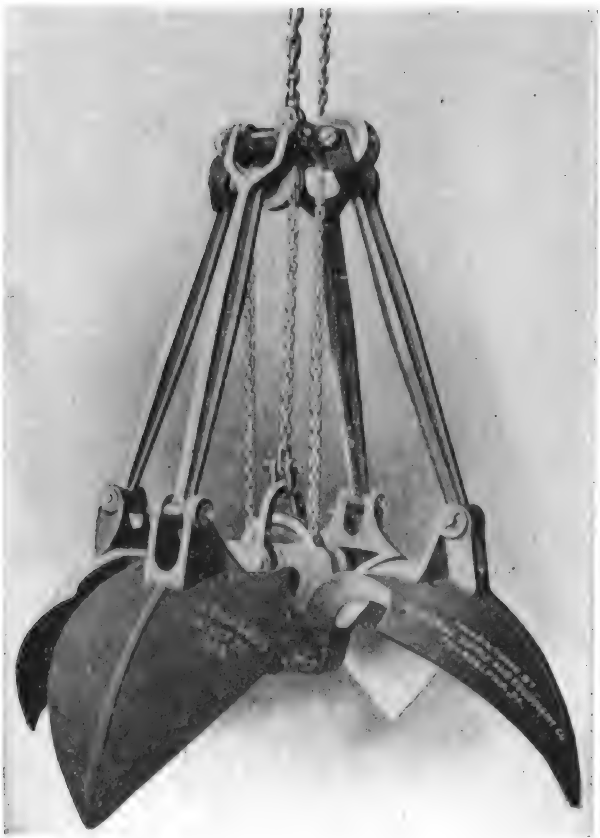


FIG. 131.—RICKARDS ORANGE PEEL BUCKET.

would be required the cost need not be greatly in excess of the above. It is usual, however, as the amount to be dredged will cost such a small proportion of the total cost of the substructure, to figure from $2 to $4 per yard for excavation in ordinary coffer-dams.

Reference has already been made to hand dredging, and a very cheap and effective scraper was illustrated in Fig. 11. Where

dredging is to be done in tubes, wells, or puddle-chambers, it can be done by a clam-shell dredge or grapple such as was shown in Fig. 57, in use on the Hawkesbury foundations.

The Lancaster bucket (Fig. 130) is a well-known form of this type of machine, and can be operated from an ordinary derrick which is served by a double-drum hoisting-engine. This dredge

FIG. 132.—RICKARDS ORANGE PEEL BUCKET.

will work best, of course, where there is some depth of soft material to be removed. While a large dredge would generally be hired by a contractor, these buckets can be owned by him and the work carried on cheaply and conveniently.

The Rickards cast-steel orange-peel buckets are of a type that is first-class for use where there is more or less loose rock to pick up, and where extra strength is required in the blades and arms.

FIG. 133.—OWEN CLAM SHELL BUCKET.

This bucket is shown in Fig. 131, and the sizes, weights, and 1914 prices in Table XIX. Where the material is of a uniform

FIG. 134.—WILLIAMS HERCULES FOUR-PART CLAM SHELL BUCKET.

nature, running from mud and sand to stiff clay or gravel, some of the clam-shell buckets that have extra interior tackle or levers for

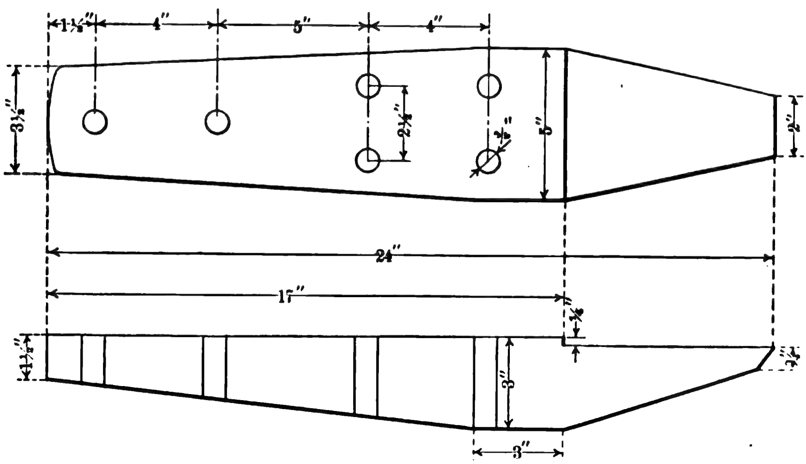


FIG. 135.—CLAM SHELL BUCKET TOOTH.

giving a powerful closing force, to close the bucket before it starts to lift off from the bottom, are best used. Some of these buckets

will do excellent work in very hard clay and packed gravel, but will not work very satisfactorily in hardpan and cemented gravel, except for the larger sizes which have weight enough to make them take hold.

The Owen bucket, Table No. XX, is one of this type, having a seven-part tackle between the top and center shaft, as shown in Fig. 133, to close the bucket. Should it be desired to use this in extra hard material, it should be built of special extra heavy design with extra large pins and bearings, and extra large riveted connections everywhere.

TABLE XIX.—RICKARDS CAST-STEEL ORANGE PEEL BUCKETS

Capacity.	Approx. Weight in Pounds.	Price.	Capacity.	Approx. Weight in Pounds.	Price.
1 cu. ft.	125	$175	1¼ cu. yds.	4300	$1200
2½ "	450	250	1½ "	4800	1350
5 "	800	450	1½ "	7000	1600
7 "	900	475	2 "	8000	1750
9 "	1000	500	2½ "	9000	1950
12 "	1600	725	3 "	10000	2150
15 "	1800	775	4 "	15000	3600
21 "	2400	1000	5 "	17000	4000
1 cu. yd.	4000	1050	6 "	20000	4400
32 cu. ft.	4100	1150	8 "	23000	5200
			10 "	27000	6000

TABLE XX.—OWEN BUCKETS

DATA ON TYPE "G" ORDINARY WEIGHT

Capacity Cu. Yds.	Weight.			Extra Jackets, Pounds.	Open.				Width. Ft. In.		Closed.			
	Bucket Less Jackets, Pounds.	Inside Jackets, Pounds.	Outside Jackets, Pounds.		Height. Ft.	In.	Length. Ft.	In.	Ft.	In.	Height. Ft.	In.	Length. Ft.	In.
½	2575	200	225	300	7	0	5	2	3	0	6	0	4	0
¾	2975	200	225	300	8	2	5	10	3	0	6	10	5	0
1	3175	200	225	300	8	10	6	6	3	0	7	2	5	8
1¼	3575	200	225	300	9	8	7	3	3	0	7	6	6	6
1½	4475	250	325	420	9	6	7	3	4	0	7	10	5	10
2	5425	250	325	420	10	8	8	2	4	0	8	7	6	10
2½	6525	250	325	420	11	6	9	0	4	0	8	8	8	0
3	7650	275	375	480	11	10	9	6	4	6	9	0	8	6
DATA ON TYPE "H" EXTRA HEAVY														
1½	5275	250	325	420	9	6	7	3	4	0	7	10	5	10
2	5795	250	325	420	10	8	8	2	4	0	8	7	6	10
2½	6625	250	325	420	11	6	9	0	4	0	8	8	8	10
3	7650	275	375	480	11	10	9	6	4	6	9	0	8	6

The Williams buckets (Fig. 134), especially of the Hercules type, are also very satisfactory, but the same criticism should be applied to them in regard to being built extra heavy for hard digging.

The teeth that come on any of these buckets from the factory are practically of no use at all, and should be replaced in the first instance with heavier and better designed steel teeth. The design shown in Fig. 135, is one which has given excellent service. The use of these buckets will be further described in Chapter XVI, under the discussion of clam-shell dredging.

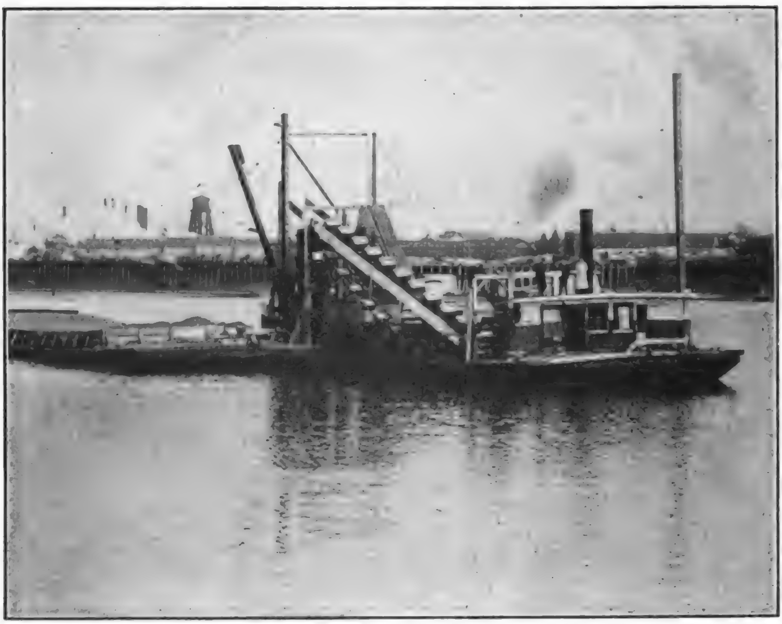


FIG. 136.—ELEVATOR SAND-DIGGER.

Sand-diggers such as were mentioned in Chapter II can often be hired where other means are not at hand, or they can be rigged up very cheaply if necessary. A smaller one than Fig. 220 can be built on a common barge, the engine being an ordinary one with a vertical boiler, while the buckets are mounted in a very simple manner and operated through a well in the center of the boat. Such a dredge will dig about 100 yards of sand per day, with only two men to attend it, and will use less than one-half ton of cheap coal, the total cost per yard thus running below five cents. Large elevator dredges of this type are very elaborate affairs, and as they are in wide use they can often be hired for making excavations.

CHAPTER X

THE FOUNDATION

THE coffer-dam is only the means of reaching a desired end, and this must be borne in mind and the construction made as simply as possible to obtain a first-class foundation.

When the coffer-dam is completed and pumped out, work can then proceed if the pumps are able to control the water easily. The character of the foundation having been previously decided upon, after a careful examination of the site, it is assumed that the temporary work has been executed in a manner which is properly related to the permanent structure.

The different kinds of bottom likely to be encountered are: First, light sand and gravel or mud of unknown depth; second, similar material overlying either cemented gravel, clay, hardpan, or rock; third, a clean rock bottom, which is approximately smooth and level; fourth, a sloping rock bottom, which is either smooth or rough, and fifth, a rough and irregular rock bottom.

Should the bottom be of the first kind—light sand and gravel or mud of unknown depth—the soft upper layer may be removed by a dredge previous to the building of the dam, or it may be removed by a dredge or grapple from within the inclosed area, and without the necessity of keeping the dam pumped out, or pumping may be kept up with a dredging-pump and the light material removed in this way, after which the heavier material may be removed as deep as necessary by hand-shoveling and a dirt-box, as shown in Fig. 98. In such a bottom the foundation is usually made by driving piles from 2 to 4 feet centers, this distance being regulated by the bearing power, as determined from Wellington's formula in Chapter IV, and building upon the tops of the piles, after they have been cut off to a level below low water, a grillage of timber. The space between the piles should be filled with broken stone or concrete, and the grillage placed entirely below low water, the coffer-dam being kept pumped out to allow this work to be

done, and also during the laying of the footing courses of the masonry which are below the water.

When the soft bottom overlays good clay, hardpan, or rock, as in the second case, and the depth exceeds 20 or 25 feet below the water-surface, piles may be driven to the harder substratum to act as bearing-piles. But when the depth is in the region of 20 feet or less, it is best to excavate and place the foundation masonry directly upon the solid bottom. The foundation will be of the character described for some of the following cases.

The third class is similar to the foundation at Chattanooga after the gravel was removed. The fissures in the rock are filled up or closed with cement and concrete, and a leveling course of concrete put down on which to found the pier (Fig. 93).

Bottoms of the fourth class should have all the loose and decomposed rock removed and steps cut out by blasting and wedging, to give a secure hold for the foundation, but if it is simply rough and irregular a leveling course of concrete will be all that is required on which to start the pier. Bottoms of clay and hardpan will require a similar treatment, except that the leveling course of concrete must be made of sufficient thickness to properly distribute the pressure, which will seldom be less than 3 feet and can often be increased with advantage to 6 or 8 feet. An example of the stepping of rock bottom was given in the account of the Forth bridge piers in Volume II, and was shown by the dotted lines.

Where there is a current caused by leakage through the sides of a coffer-dam, or from the bottom, or if the water within the dam is agitated by the pumping, it will be best, after the bottom is clean and properly prepared, to allow the water to run in and then deposit the concrete through the still water. This has been successfully accomplished through 25 or 30 feet of water, and while some engineers recommend allowing the concrete to set from one to three hours before depositing, to prevent the cement from washing out of the concrete, this is not necessary or advisable if the proper care is exercised and the proper apparatus used. The concrete should be made from one-third to one-half richer than would be used for similar open-air work, as there will be some loss of strength.

The simplest method is to deposit the concrete in paper sacks by sliding them down a smooth wooden or iron chute, or by loading them into a box or skip and dumping them out after the box reaches the bottom. The sacks should be of tough paper, similar to flour sacks, and when they reach the bottom they may be broken by a pike-pole and the concrete allowed to run together. Thin

cloth sacks are sometimes used and they become fairly well cemented together by the mortar which oozes through.

Where the amount of concrete is considerable it will be best to use a tube or bottom-dumping box. The use of a steel pipe *trémie* in the most approved manner is described in Volume II. For placing concrete under water on the Boucicault bridge over the river Saône in France a wooden tube 16 inches square was used. This is described in the *Engineering News* of May 18, 1893. The tube was carried transversely across the caisson on a traveling-crane which ran lengthwise of the caisson on tracks on the sides, thus allowing the tube to be moved in any desired direction. The tube was built in sections which could be easily removed, was provided with a hopper at the top into which the concrete was dumped, and a drop-door at the bottom to let out the concrete. The tube was filled as it was lowered down into the water, and opened when within 16 inches of the bottom. As concrete was dumped in above, the tube was moved about and a 16-inch layer of concrete deposited. When one layer was complete, another of the same thickness was deposited. This method of using 16-inch layers was said to have obviated laitance or the exuding of the gelatinous fluid which prevents uniform setting. The concrete was deposited about the heads of the piles and no grillage used. The thickness of the concrete, which was deposited at the rate of from 90 to 100 yards per day, was 9.84 feet, and was allowed to set fourteen days before the pier was begun.

A metal tube may be used, such as was employed on the Harvard bridge at Boston by W. H. Ward. This tube (Fig. 137) was not provided with a bottom and the first filling of the tube was consequently done after the tube was lowered and the concrete became somewhat washed. This may easily be prevented by using concrete in paper sacks to fill the tube the first time. The tube was suspended from a derrick and was moved about so as to keep the concrete level and deposit it in layers. This account is taken from Vol. 31 of the *Engineering News*, from which is taken the following description of a metal bucket used by W. D. Taylor on the Coosa River.

This bucket (Fig. 138) was of riveted construction and held one yard of concrete. The maximum depth of water was 26 feet, at which depth the bucket and its load became so lightened that the bucket tripped as soon as the flanges touched the bottom. Similar boxes are often constructed of wood, or they are often made V-shaped, one side being arranged to open and dump the load. The placing of concrete in the dry by a bottom-dumping bucket is described for the Tacoma bridge, Volume II.

For concrete work of this character natural cement is often used, but on all important work Portland cement should be employed.

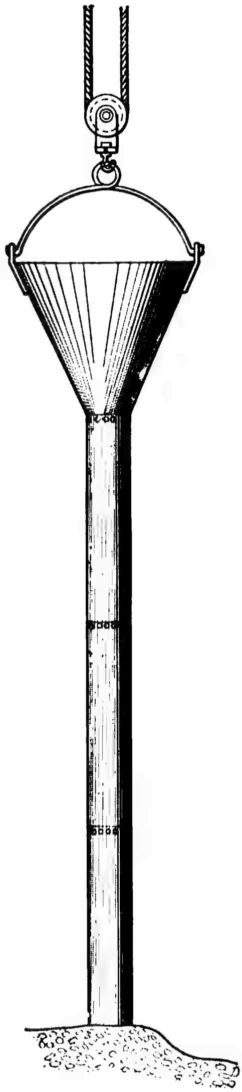


FIG. 137.—METAL TUBE FOR CONCRETING.

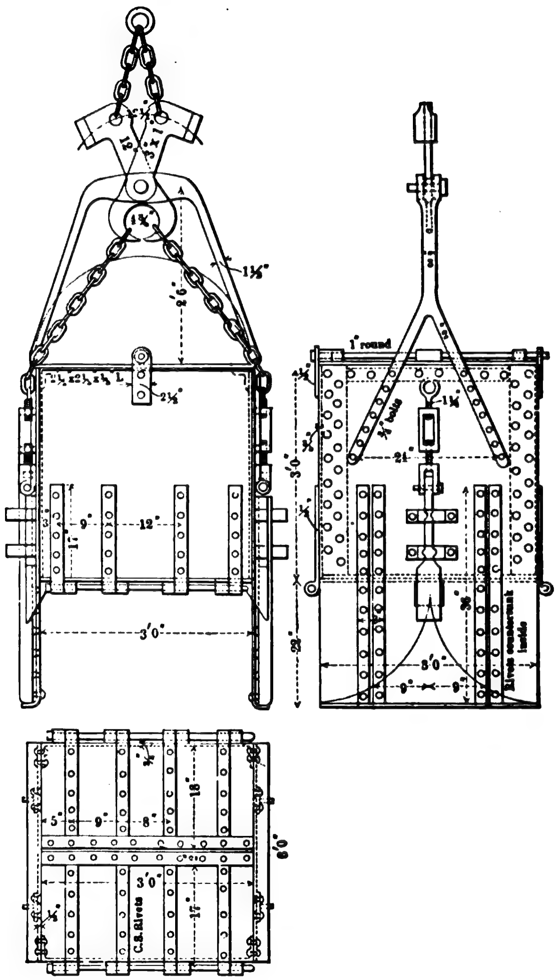


FIG. 138.—METAL BUCKET FOR CONCRETING.

The proportions range from one of cement, two of sand, and four of broken stone, to one of cement, three of sand, and six of broken stone.

On such a base either a masonry or monolithic concrete pier may be constructed. The pier at Little Rock (Fig. 96) was of this construction and of the composition given in Chapter VII. A similar piece of work was constructed on the Red River bridge on the St. Louis & San Francisco Railway and is described in the *Engineering News* of June 2, 1888, by C. D. Purdon, assistant engineer

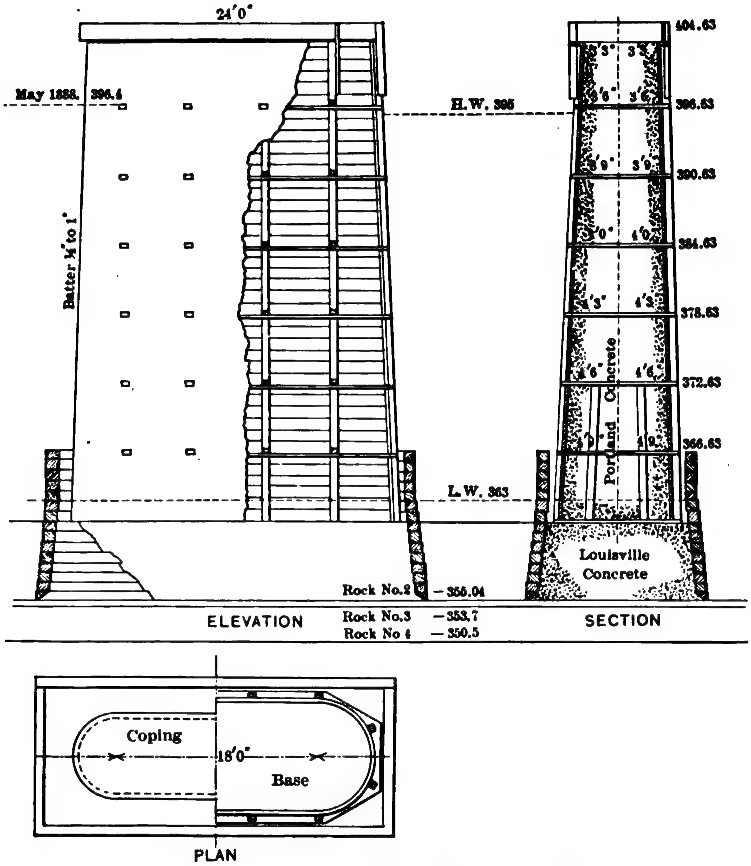


FIG. 139.—CONCRETE PIERS, RED RIVER BRIDGE.

in charge of the work under James Dun, chief engineer. The cribs were filled with Louisville cement concrete up to within 2 feet of low water, on which was built the pier. (Figs. 139 and 140.)

"After the crib had been filled with concrete and the surface leveled off, the center lines of the pier were located and a frame of 2×8-inch plank, the shape of the pier, and 4 inches larger to allow

for lagging, was placed in exact position and held by pieces spiked to the crib. On this frame upright posts 6×6 inches and 5 feet 10 inches high, with a batter of ½ inch per 3 feet, were set in the position shown on the drawing, then the feet spiked to the frame and another frame similar to the first, but 6 inches narrower, placed on them. This again was brought to exact position and braced to the crib and the frame completed by putting lagging of 2-inch

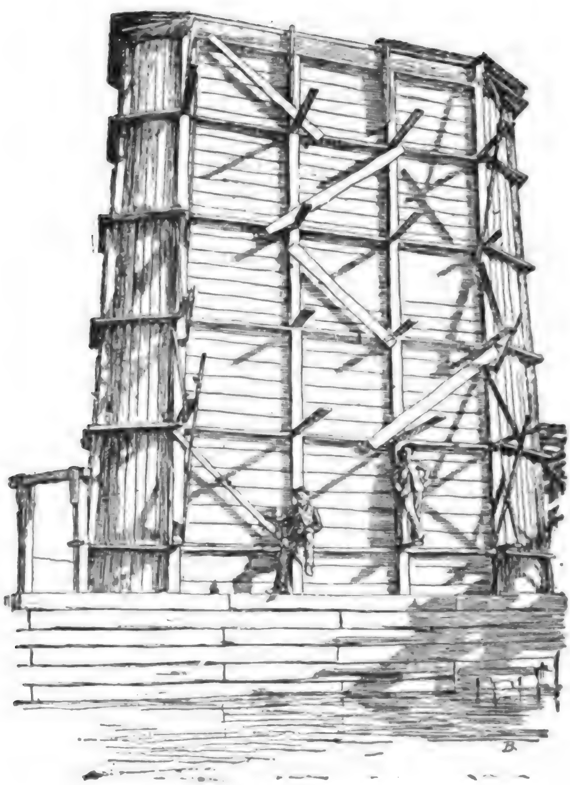


FIG. 140.—CONCRETE FORMS, RED RIVER BRIDGE.

plank inside the posts and spiking to them. This lagging was horizontal in the body of the pier and vertical (2×4 inches) at the ends, beveled pieces being introduced in the ends at intervals to make up the difference of the upper and lower circles. Next 2×6-inch planks were placed across on the tops of the posts, running clear through the pier, to act as braces. In the rest of the frames these braces were allowed to extend about 6 feet on each side and

the frame braced by spiking plank to them and to the vertical posts. After a section of frames was completed a bed of cement mortar about 2 inches thick was spread all over the concrete in the crib. On this rough stone, in such pieces as one man could easily handle, was placed so that no two pieces would be closer than 2 inches, nor any piece within 2 inches of the frame, the stone being thoroughly wet before laying.

" Next, on this course of stone another bed of mortar was placed, sufficient to fill all the spaces between the stones and remain about 2 inches thick above them. It was then well rammed with rammers made by inserting a handle in a section of a pile, except at the edges, where a rammer made of a 2-inch plank cut in the shape of a spade was used, to insure a perfect skin of cement without any breaks. After this had been well rammed, another layer of stone was placed and covered with mortar as before, and so on.

" The coping, which was made similar to the body of the pier, was finished by about 1½ inches of cement mixed with sand one to one, fluid enough to be struck off by a straightedge, the top of the frame being dressed and leveled for that purpose.

" After the pier had been completed the frames were removed and the braces running through the piers cut off by a chisel inside the concrete. Then, to make a smooth surface, the pier was thoroughly wet and plastered with a mixture of 1 part sand to 1 part cement, after all the rough or loose portions had been scraped off. This was mainly done for appearance."

The mortar for the body of the pier was made of 1 part Alsen's German Portland cement and 4 parts of sand. There was used about 1⅓ barrels of cement to a cubic yard of completed pier. In mixing the mortar eleven ordinary pails full of water were used to one barrel of cement, which caused the water to just appear on the surface when the tamping was done.

The lock walls on the Illinois and Mississippi canal have been constructed of monolithic concrete under Captain W. L. Marshall, Corps of Engineers. The work was executed under L. L. Wheeler, engineer in charge, from whose account, in the report of the Chief of Engineers for 1894, the following is taken:

" The rules adopted for the work were adhered to and are worthy of careful study.

" I. The forms or molds of the walls will be divided by vertical partitions perpendicular to the longest axis of the mass, and the walls be constructed by filling alternate sections.

" II. The sections will be filled in horizontal layers, well rammed,

each layer to be deposited before the 'initial set' of the previously deposited layer. When the work of filling a section is begun it must proceed without intermission to completion, working night and day if necessary.

"III. The facing and backing must go on simultaneously in the same horizontal layers, using the same cement in the facing as in the backing, with no defined lines of demarcation between the facing which contains no stone and the concrete backing.

"IV. When the top surface of the coping is reached it will be finished after ramming by cutting off the excess by a straightedge, and rubbed smooth and hard by a float. No plastering or wet finishing will be allowed.

"V. The facing of the walls will not be finished by plastering or washing with cement after the forms are removed, nor dressed in any manner beyond chiseling away rough ridges, should the plank forming not be smooth.

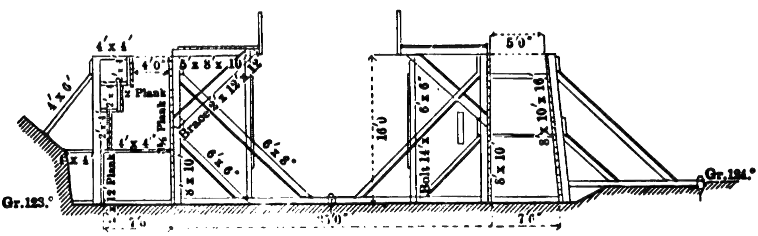


FIG. 141.—CONCRETE FORMS, ILLINOIS AND MICHIGAN CANAL.

"VI. The concrete shall be mixed with all the water it will take without water showing after ramming, or without 'quaking' upon ramming.

"VII. At such intervals as may be necessary vertical wells, at least 1 foot square, will be formed along the middle of the masses of concrete, reaching to near the bottom thereof. The masses of concrete after forming will be kept sheltered from the sun, the outer surfaces kept moist and the wells kept filled with water until well set, or about three weeks. The walls will then be filled with concrete.

"VIII. In preparing the cement for mixing with other ingredients of concrete, from five to ten barrels will be kept thoroughly mixed dry, to guard against chance barrels of defective cement, and the necessary quantity of cement will be taken for each batch from this mixture.

"IX. Two cements of different qualities shall not be used in the same section, but as far as practicable each mass shall be homo-

geneous throughout, but a slight excess of cement in the facing to reduce its capacity to absorb water."

The rate at which the concrete was deposited in the work was determined by the rate of ramming, and but one yard every five minutes was deposited. The forms (Fig. 141) were lined with dressed pine plank 4×8 inches on the face, of uniform thickness, and with 2-inch rough plank on the back.

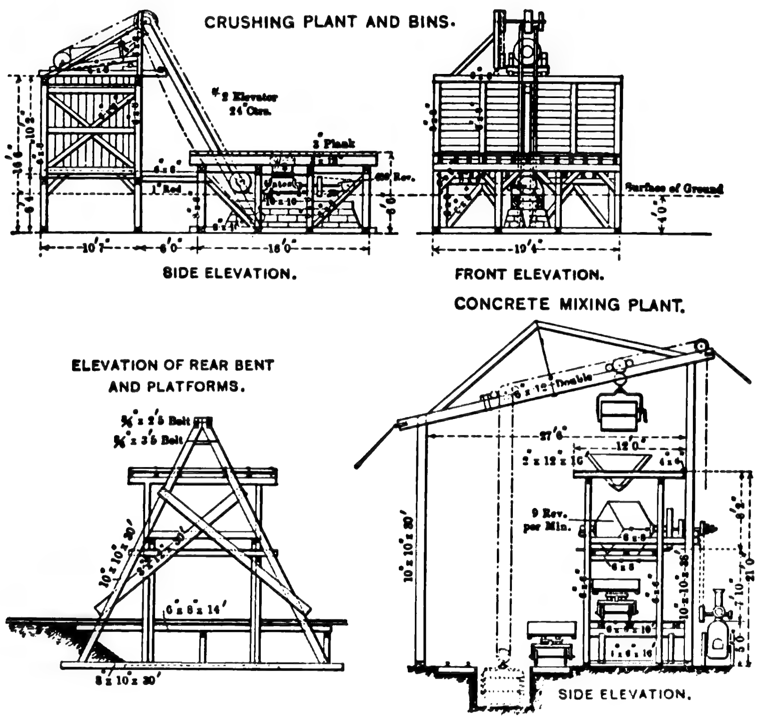


Fig. 142.—Stone Crusher and Concrete Mixer, Illinois and Michigan Canal.

Rough plank is sometimes used on such work and lined with oiled paper, or ordinary dressed plank may be used and coated with soft soap. In most sections of the country crushed broken stone can be obtained, but owing to the magnitude of this work a crusher was built (Fig. 142) and was found to work very satisfactorily. The concrete mixer shown in Fig. 142 was operated by a 15-horse-power portable engine. The proportions finally adopted for the concrete were 1 of cement, $3\frac{1}{3}$ of gravel, and 4 of broken stone, while the facing and coping was composed of 1 part cement and 2 parts of clean river sand.

That the sand for concrete be clean and sharp is very essential, and any loam or dirt must be washed out. Equally essential is good, clean, sharp, broken stone without dust or dirt. The cement used on the above work was a German Portland, but most of the American Portlands are first class and will give as good results as the imported.

Where good, fresh, cement is being supplied a few tests to a car-load will be sufficient, and for cements of the grade of Atlas or

FIG. 143.—REINFORCING BAR BENDER.

Empire the guarantee of the manufacturer, supplemented by a few tests, should be sufficient. But for cements which have been shipped by water, tests should be made from every five or ten barrels.

The Atlas Cement Company recommend, for concrete laid in open air on moist ground where great weight must be carried, 1 of cement, 2 of clean sharp sand, and 4 of 2-inch broken stone; this sand and cement to be thoroughly mixed dry, then just enough water added to thoroughly moisten, and the mass turned over at

least twice, when the stone is to be added in a thoroughly wet condition. This must then be put at once into the molds and well rammed.

Where a solid bottom is to be built upon, the proportions of 1 of cement, 3 of sand, and 6 of broken stone are recommended. For ordinary construction 1 of cement, 4 of sand, and 8 of broken stone, while to obtain a concrete as strong as ordinary natural

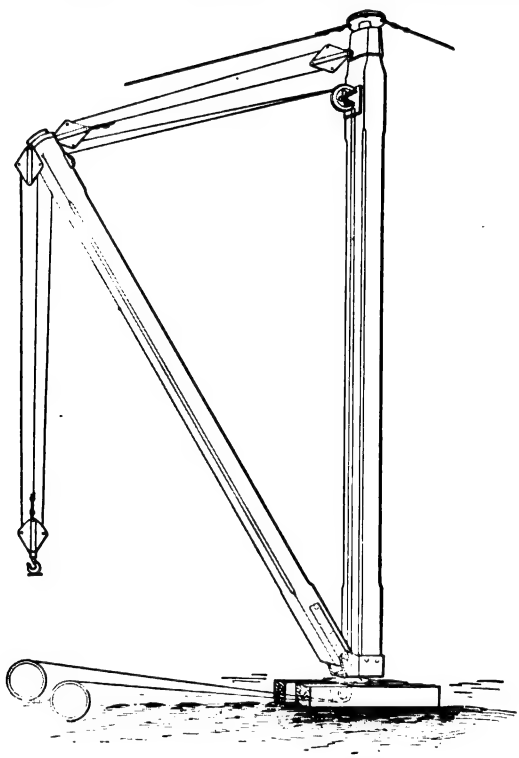


FIG. 144.—DOUBLE-DRUM GUY DERRICK, AMERICAN HOIST AND DERRICK CO.

cement concrete, 1 of cement, 5 of sand, and 10 of broken stone can be used.

The average cost in the western portion of the United States of such concretes, including labor, tools, timber forms, and a fair profit to the contractor, would be for the first \$8 per cubic yard, for the second \$7.50, for the third \$7.25, and for the fourth \$7. In the Eastern States they can be figured at \$1 per cubic yard less.

Where reinforcing is to be used in foundation concrete it is very often necessary to bend the reinforcing bars. A tool for this purpose,

such as is shown in Fig. 291, will bend up to 1¼-inch high-carbon bars without heating. It can be operated by one man, by the ratchet lever, and the dies can be adjusted for the thickness of the steel to be bent by means of the set-screws. The machine weighs about 350 pounds, and costs $100.

Where the leveling course of concrete has been put in and the pier is to be of stone, the footing course should be of carefully selected material. They should be large stones with good beds, and should be as thick or preferably thicker than the courses above. Where the bearing pressure does not exceed two tons per square foot, the footing courses may be stepped by allowing each course

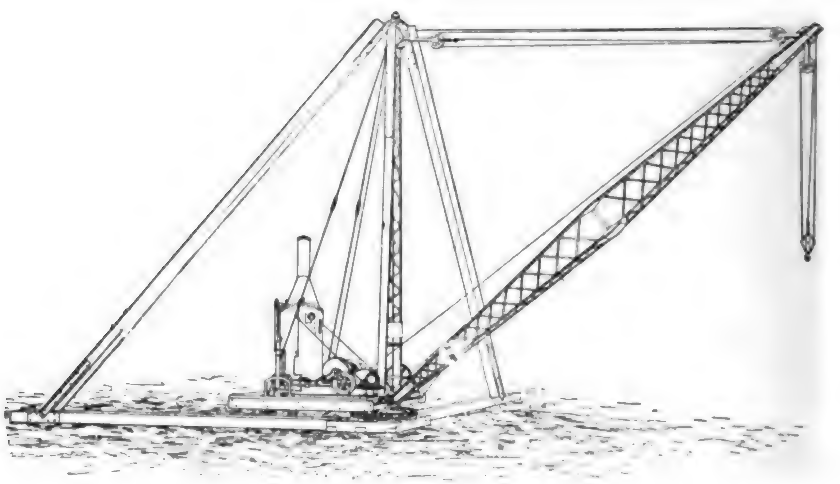


FIG. 145.—STIFF-LEG DERRICK WITH STEEL BOOM.

to project about one and one-third times its thickness, depending of course on the quality of the stone.

The usual way of handling the material for foundations and piers is to boat it to the site, where it is placed by a stiff-leg derrick, or, if guys can be used, by a derrick with wire-rope guys.

The stiff-leg derrick shown in Fig. 145 is fitted with steel mast and boom, but where timber is plenty the entire rig can be made with large timbers similar to the stiff-leg derrick on the scows, shown in Volume II, and the author has used fir booms in one stick up to 90 feet long, as shown in Fig. 105. Everything about a stiff-leg derrick must be carefully proportioned, including the goose necks and all of the connection plates and pins.

Derrick lines and other hoisting lines can be handled by crabs or winches. A style of which used by the author for many years is shown in Fig. 146 in full detail, and can be easily constructed by ordinary first-class workmen.

The fittings for such derricks can be obtained from a number of firms, an American Hoist and Derrick Company outfit being shown in Fig. 144. This is rigged to be operated by a double-drum hoist, which can be one operated by horse-power (Fig. 147) if the piers are near the bank and if steam-power is not available. The usual form, however, is a double-drum steam-hoist like the Lidgerwood machine shown in Fig. 148.

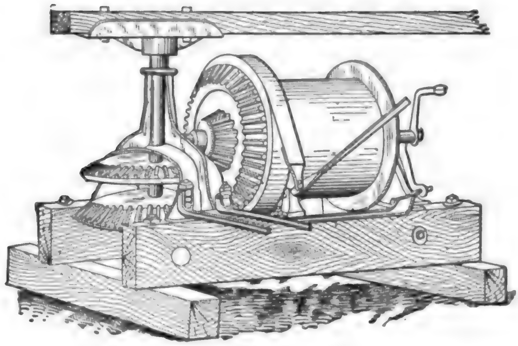


Fig 147.—Single-drum Horse-power, Contractors' Plant Mfg. Co.

Instead of swinging a derrick by hand lines or by tag lines running through snatch blocks to the nigger heads, it is best to employ slewing engines similar to the one shown in Fig. 149, which can be operated by the hoisting engineer, and save one or two men on the work. This kind of a rig used in connection with a bull-wheel on the derrick will pay for itself usually in two or three months' time. Hoist engines fitted with two reversing drums for swinging the boom are all similar to the engine shown in Fig. 150. This style of an engine will work well on land, but not so well on a scow, which is apt to list and cause an especially hard pull. The sizes and data of hoist engines are shown in Tables XXI and XXII.

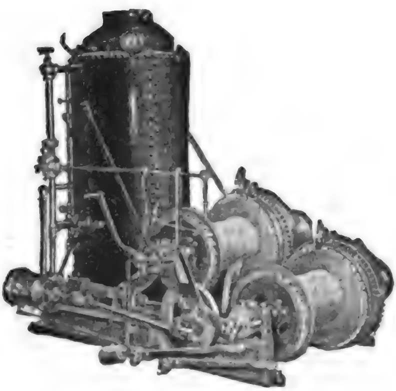


Fig. 148. — Double-drum Hoist Engine, Lidgerwood Mfg. Co.

Vertical boilers are often required about a contractor's plant similar to the one illustrated in Fig. 151, and are fully described in Table XXIII. All boilers should be built to comply with the most

rigid requirements of City Boiler Inspection Ordinances, and one somewhat exceeding these requirements, built under Government Navy specifications, is shown in Fig. 152. This boiler, however, is of such a size that it should not be used for small construction, owing to the expense of moving and setting it up, and boilers of

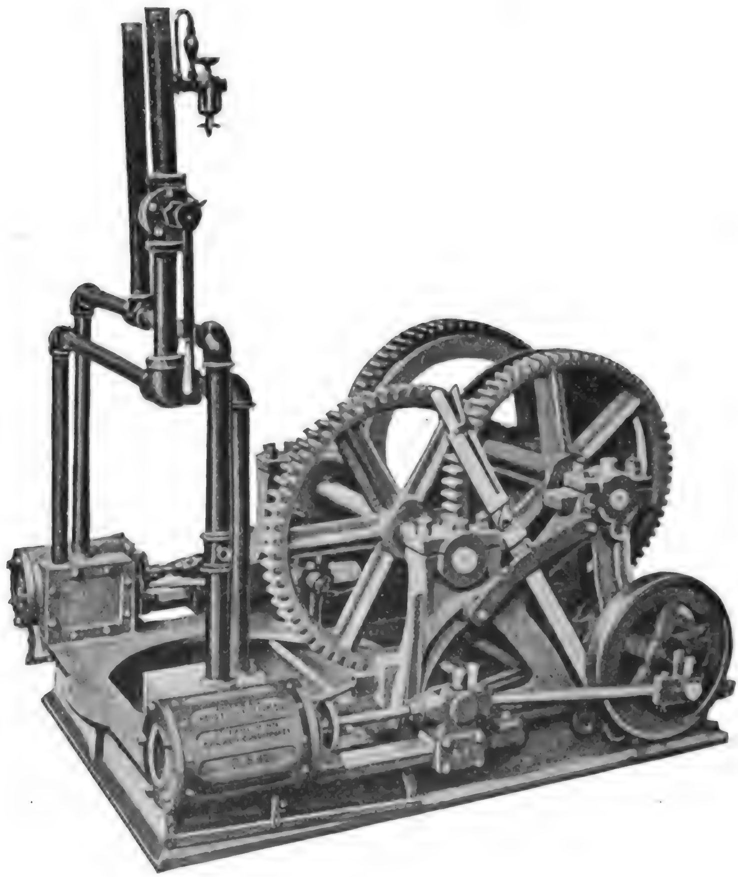


FIG. 149.—SLEWING ENGINE FOR DERRICK.

the locomotive type described in Chapter V, are much more easily transported and placed.

Where electric power is available an electric hoist (Fig. 153) should be used, as it will be found much more convenient.

Works of any magnitude should, however, be fitted from the beginning with a cableway, which will avoid the necessity of boat-

ing the materials, erecting of large derricks, and facilitate in every way the prosecution of the work, besides often making a balance on the right side of the ledger. The Lidgerwood cableway on dam No. 11 of the Great Kanawha River, a tower of which can be seen

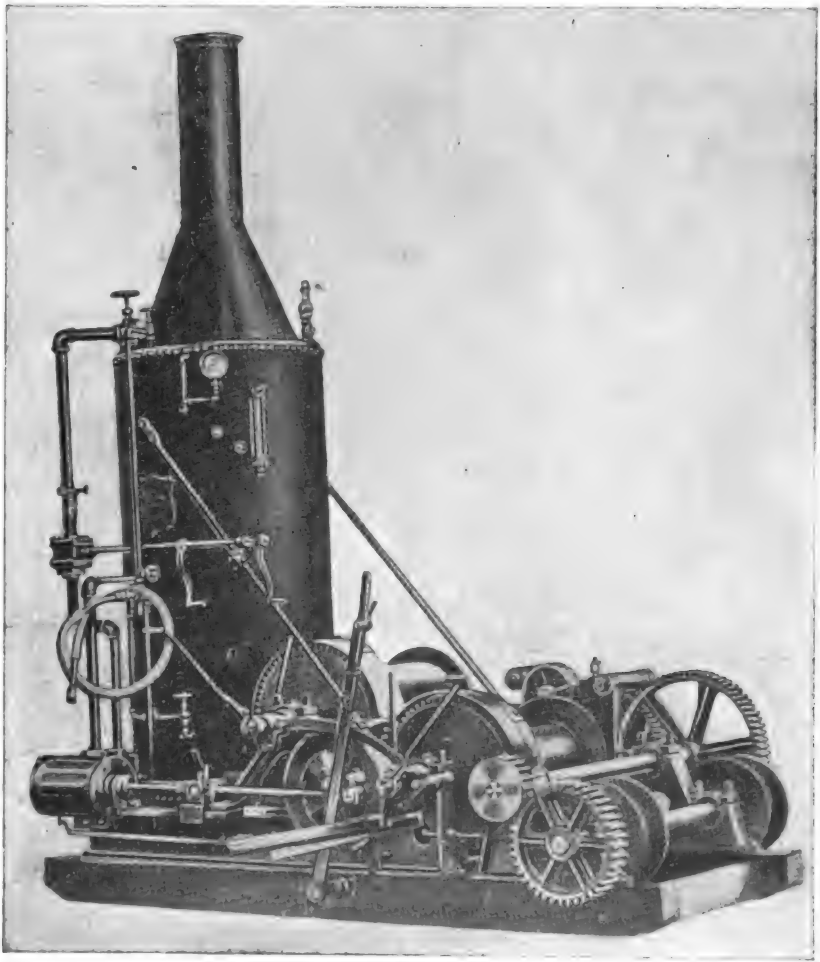


FIG. 150.—DERRICK ENGINE WITH TWO REVERSING DRUMS FOR SLEWING BOOM.

in Fig. 12, had a span of 1505.5 feet and carried a net load of four tons on a main cable $2\frac{1}{2}$ inches in diameter. The stone quarry was located on one bank, and the stone was taken directly to the stone-yard and to the work in the river. A seam of coal in the quarry also

supplied fuel for the dredges and pumps, the coal being handled by the cableway, as was also the material from the railroad siding on the opposite bank.

The details of these cableways have been developed and perfected to a wonderful extent, as a result of their use on the Chicago drainage

FIG. 151.—VERTICAL BOILER.

channel. The engine for operating one of these with a capacity of eight tons has double 10×12-inch cylinders, the cranks being set at an angle of 90°, and is provided with reversing link motion. The double drums regulate both the hoist at a speed of 300 feet per minute and the travel along the cable at 1000 feet per minute. A 70-horse-power boiler is required.

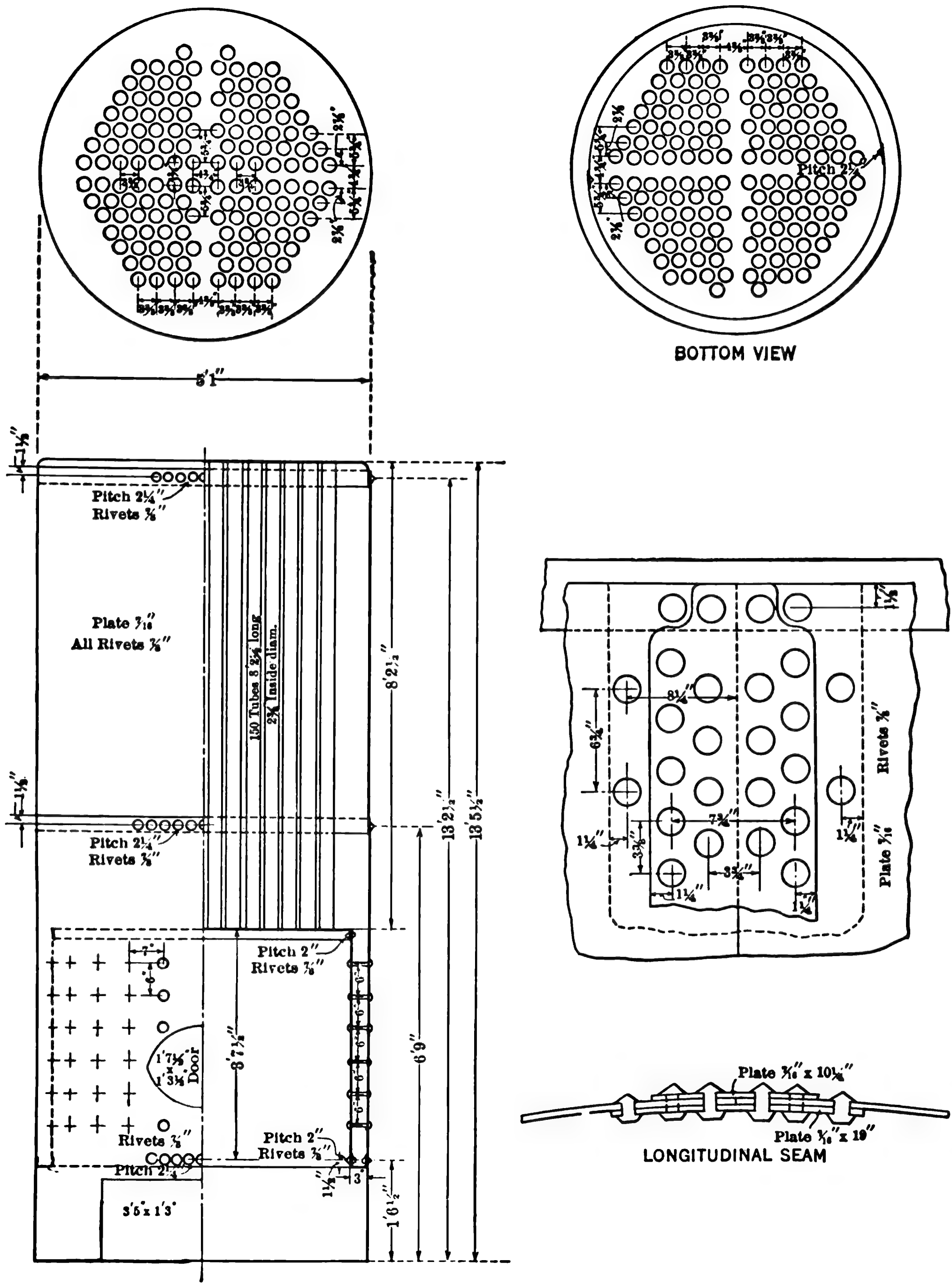


Fig. 152.—Vertical Boiler, Navy Type.

The carriage and skip, which are automatic in action, are shown in Fig. 154, the capacity of those on the drainage channel being 1.8 yards, and the average of a month being about 600 yards per day of ten hours. The cost of operation, including labor, fuel, and everything except interest on plant and repairs, was less than $18 per day or from 3 to 4 cents per yard.

The cableway on the Coosa dam and lock (Fig. 155) had a capacity of about eight tons and made a round trip on an average of about three minutes. Such a plant is out of reach of high water and of trains where used over railroad tracks as at the North Avenue bridge in Baltimore.

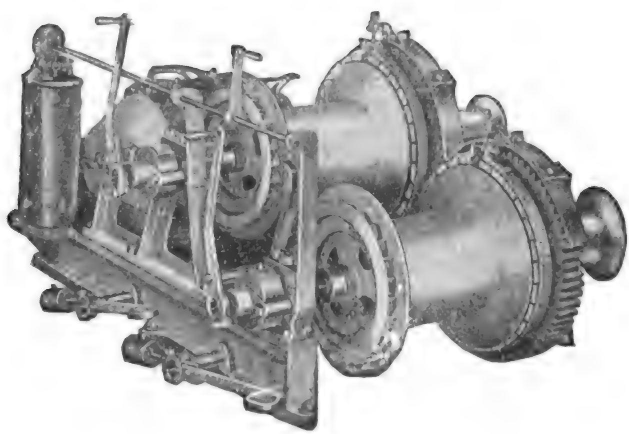


FIG. 153.—LIDGERWOOD ELECTRIC HOIST.

The Court Street stone-arch bridge at Rochester, N. Y., of eight spans, was constructed with the aid of a cableway, which was also used to remove the old bridge and piers. A cableway of one span was used to construct the Melan concrete-arch bridge at Topeka, Kan. The bridge has five spans and a total length of 650 feet. During the extreme high water in the early part of 1897, when everything was completely inundated, and an ordinary derrick plant would have been swept away, the cableway was high and dry out of reach of the flood.

The prevailing low prices of contract work make it necessary to employ every improvement on important engineering work, and the

cableway has doubtless come to stay as one of the most remarkable of our tools.

Very often it becomes necessary to use jacks of large capacity around foundation work, and while the hydraulic jack is very satisfactory when it is in good shape and in warm weather, the Norton jack, Fig. 156, can be purchased up to 100 tons capacity, and will usually be found much more satisfactory. This is a ball-bearing ratchet-screw jack, and worn or broken parts can be easily replaced.

They are useful around caisson work for jacking-up during the

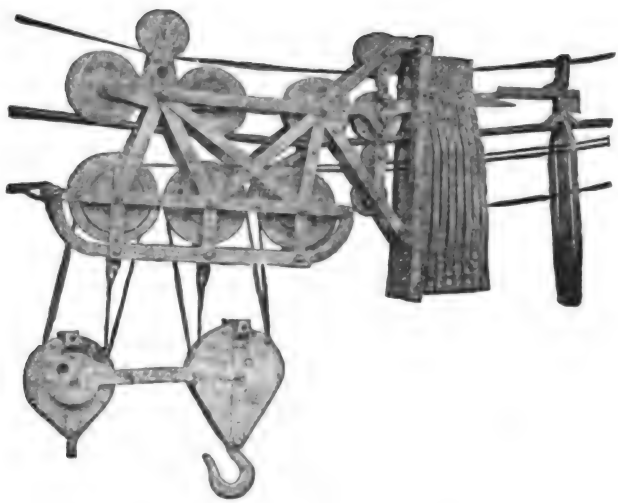


FIG. 154.—LIDGERWOOD HIGH-SPEED CABLEWAY CARRIAGE AND FALL ROPE CARRIERS.

building of the cribs and in launching them when they must be started with one or more jacks.

After caissons are in place, it is often necessary to use jacks to loosen up braces, so they can be removed or replaced. The same work in cofferdams can be done more easily by the use of some form of jack, and, if others are not at hand, ordinary screw jacks may be employed.

The necessity for the proper lighting of foundation work where night crews are employed is one of the most important matters to be considered by the engineer, and wherever electric current can be obtained electric lights should be used instead of gasoline torches,

which are very unsatisfactory, especially in windy weather. A direct-connected DeLaval turbine generator, as shown in Fig. 157, occupies

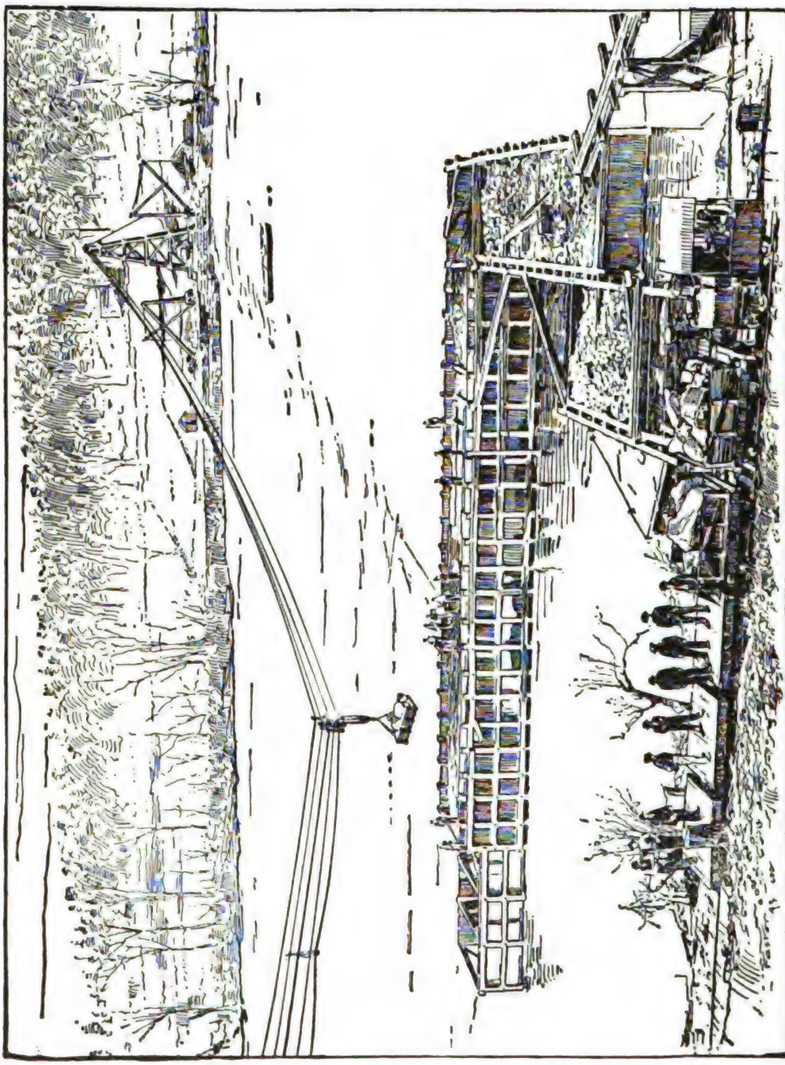


Fig. 155.—Lidgerwood Cableway at Coosa Dam. Span 1012 Feet.

but little space and can be used to advantage on any work where steam boilers are in use.

Very often it is more convenient to use an acetylene light as shown in Fig. 158, which is a Milburn portable lamp, and which can be

FIG. 156.—NORTON BALL-BEARING RATCHET SCREW JACK.

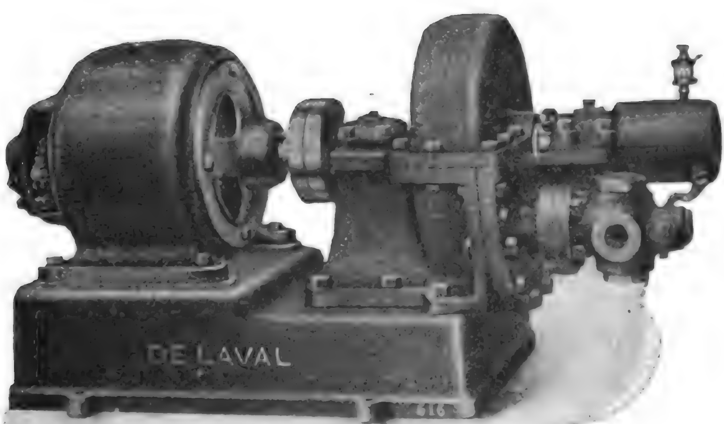


FIG. 157.—DE LAVAL TURBINE DRIVING GENERATOR.

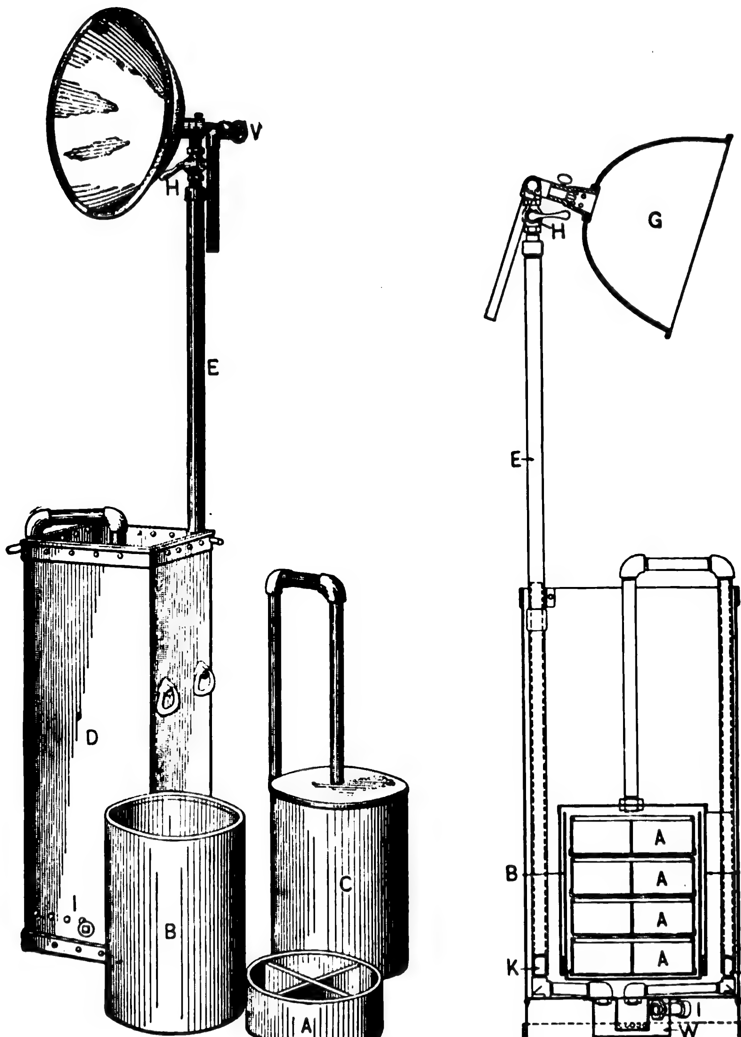


FIG. 158.—MILBURN PORTABLE LIGHT.

obtained up to 12,000 c.p. Usually one of about 1500 c.p. is all that will be required for ordinary work, and should more light be needed additional ones may be added as required.

TABLE XXI.—TABLE OF SIZES, LIDGERWOOD SINGLE-CYLINDER, SINGLE-DRUM HOISTING-ENGINES.

Horse-power Usually Rated.	Dimensions of Cylinder.		Weight Hoisted Single Rope, Usual Speed. Pounds.	Suitable Weight of Pile-driving Hammer for Quick Work. Lbs.	Dimensions of Hoisting-drum.			Dimensions of Bed-plate		Dimensions of Boiler.			Estimated Shipping Weight Complete, etc. Lbs.
	Diameter. Inches.	Stroke. Inches.			Diam. Body between Flanges, Inches.	Length Body between Flanges. Inches.	Diameter Flanges. Inches.	Width. Inches.	Length. Inches.	Diameter Shell. Inches.	Height Shell, Inches.	Number of 2-inch Tubes.	
4	5	8	1200	1000	10	20	22	38	60	28	63	40	3550
6	6¼	8	1500	1250	10	20	22	38	60	28	69	40	3950
8	6¼	10	1750	1500	12	20	24	41	73	30	72	44	4850
10	7	10	2500	1800	12	20	24	41	73	32	75	48	5050
11	7	10	2500	2000	14	22	26	45	73	34	78	52	5350
12½	8¼	10	4000	2500	14	23	29	47	73	36	75	57	6550
15	8¼	10	4000	2800	14	23	29	47	73	36	81	57	6750
20	8½	12	6000	4000	16	26	33	54	84	40	84	80	8500
25	10	12	8000	5000	16	26	33	54	84	42	90	88	9500

TABLE XXII.—TABLE OF SIZES, LIDGERWOOD DOUBLE-CYLINDER, DOUBLE-DRUM HOISTING-ENGINES.

Horse-power Usually Rated.	Dimensions of Cylinders.		Dimensions of Hoisting-drums.		Weight Hoisted Single Rope, Average Speed.	Suitable Weight of Pile-driving Hammer for Quick Work.	Dimensions of Boiler.			Dimensions of Bed-plate.		Estimated Shipping Weight with Boiler Complete.
	Diameter. Inches.	Stroke. Inches.	Diameter. Inches.	Length. Inches.			Diameter. Inches.	Height. Inches.	Number of 2-in. Tubes.	Width. Inches.	Length. Inches.	
8	5	8	12	22	2,000	1,500	32	75	48	47	80	6,500
12	6¼	8	14	22	3,000	2,000	36	75	57	50	86	8,000
16	6¼	10	14	26	4,000	2,800	38	81	68	54	89	9,000
20	7	10	14	26	5,000	3,500	40	84	80	54	89	9,550
30	8¼	10	14	27	8,000	5,000	42	90	88	57	94	11,400
40	8½	12	16	32	10,000	8,000	50	102	124	70	117	21,000
50	10	12	16	32	12,000	10,000	53	102	150	70	117	22,000

TABLE XXIII.—VERTICAL BOILERS.

Nominal Rated Horse-Power.	Shell.			Fire Box.		Mean Thickness.		Size of Steam Outlet.	Tubes.				Total Heating Surface.	Stack.		Shipping Weight Pounds Approximate.		
	Diam.	Height over all.		Diam.	Height.	Shell.	Heads.		No.	Diam.	Length.			Diam.	Length.	Boiler Only.	Fixtures Only.	Boiler and Fixtures.
	In.	Ft.	In.	In.	In.	In.	In.	In.		In.	Ft.	In.	Sq. Ft.	In.	Ft.			
6	28	5	10	$23\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{4}$	42	2	2	8	75	10	15	1209	391	1600
8	30	6	5	$25\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{2}$	48	2	3	3	99	12	15	1446	454	1900
10	30	7	4	$25\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{2}$	48	2	4	2	122	12	20	1593	507	2100
15	34	7	8	$29\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	2	70	2	4	6	186	14	25	2058	642	2700
20	36	8	9	$31\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	2	76	2	5	7	245	15	25	2645	755	3400
25	38	9	3	$33\frac{1}{2}$	27	$\frac{1}{4}$	$\frac{3}{8}$	$2\frac{1}{2}$	88	2	6	1	304	16	30	3153	947	4100

Each boiler, on cast-iron base plate, is furnished with grate bars, safety valve, steam gage and siphon, glass water gage and gage cocks, check valve, stop valve, blow-off cock, smokestack, hoods and guys.

All plate is flange steel, guaranteed by the makers to be 60,000 pounds tensile strength and to turn down double cold without fracture.

CHAPTER XI

THE FOUNDATION (CONTINUED)*

THE determination of the exact character of the foundation is entirely dependent upon the bearing capacity of the soil or foundation bed. In cases where there is large doubt as to what the bottom will carry per square foot of surface, it is always best to make some tests to arrive at definite conclusions; but in ordinary cases it is possible to adopt figures well within safe limits, and thus avoid the trouble and expense of experimenting. Unless the experiments are very carefully conducted, precedent is the better method.

For the State Capitol at Albany, N. Y., one of the most important structures in the United States, very careful and elaborate experiments were conducted by W. J. McAlpine, the engineer in charge of the work; and as the material, which was blue clay, was found to sustain a load of 6 tons per square foot, it was decided to adopt 2 tons as the safe load to put upon the foundation bed.

The Congressional Library at Washington, D. C. (Fig. 159), another very important building, had the foundation constructed to come within 2½ tons per square foot, although the yellow clay was found to carry a total load of 13½ tons per square foot.

It is always possible, of course, to thoroughly drain the foundations of a building, or to at least know the exact condition in which the foundation bed will exist; but for bridge work the circumstances are very different, and after the foundation is once in place, unless it be on solid rock, examinations are very difficult, so that it is necessary to be much more sure as to what the material will carry.

The Bismarck bridge (Fig. 160) across the Missouri River, on the line of the Northern Pacific Railway, has the piers (Fig. 161) founded upon the clay, which was found to sustain a load of 15 tons per square foot before settlement ensued, and the actual load is 3 tons per square foot. From a report on this work the following account is taken of the character of the foundation bed:

* See formula for bearing on clay, Antwerp Quay Wall, Volume II. Also Chapter XIV.

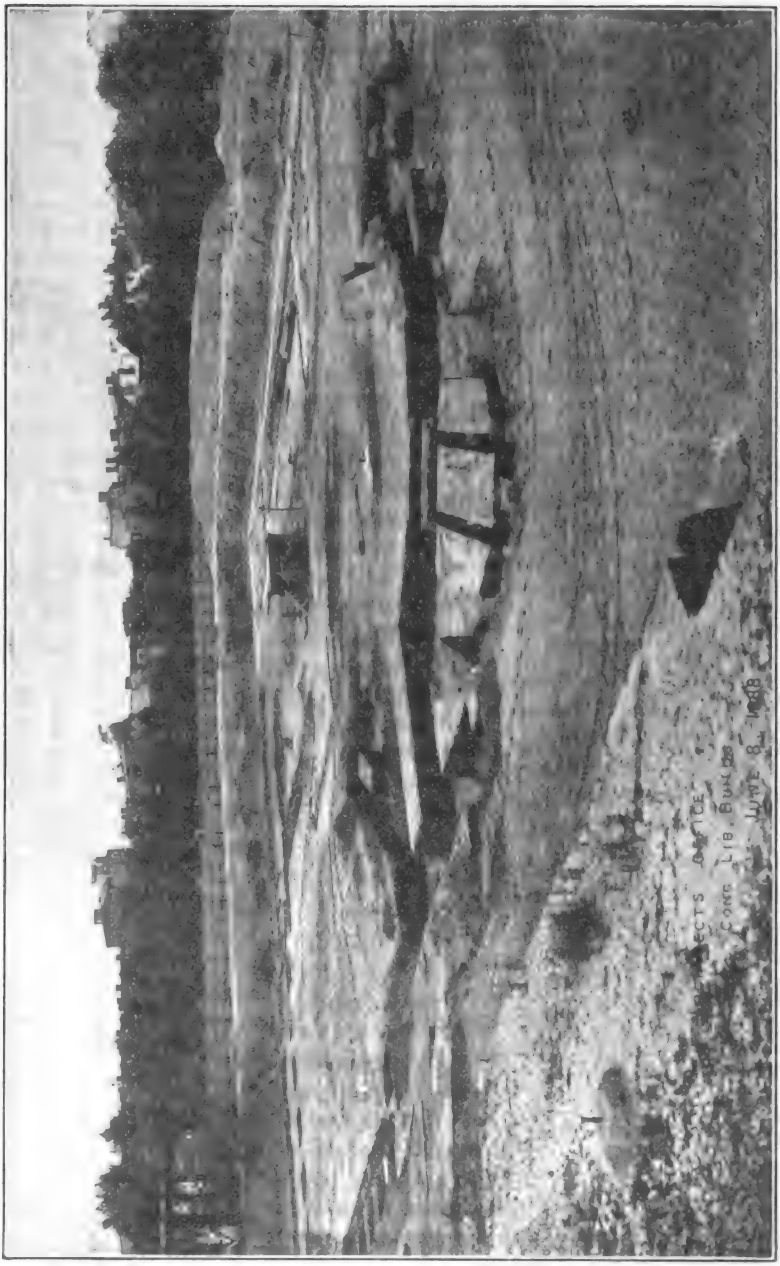


FIG. 159.—CONGRESSIONAL LIBRARY, WASHINGTON, D. C.

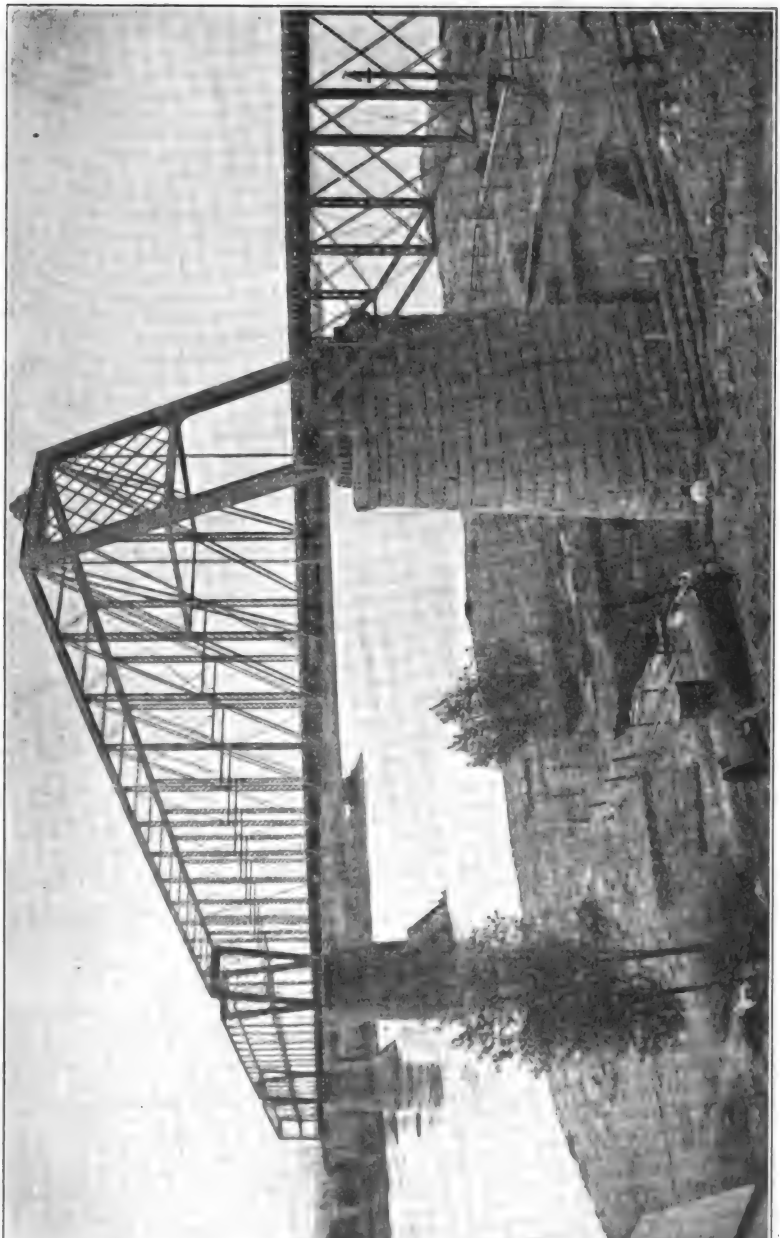


FIG. 160.—BISMARCK BRIDGE, NORTHERN PACIFIC RAILWAY.

"With the exception of some thin strata of soft sandstone of irregular thickness and extent, no rock is found in position in this part of Dakota. The entire country is underlaid with a very hard stratified clay, the depth of which has not been ascertained. Borings proved this clay to be at least 100 feet thick on the line of the bridge, and a hole intended for an artesian well has since been sunk within the Bismarck city limits to a depth of over 1300 feet in the clay. This clay, however, is in many respects more like a rock than a clay; small specimens tested for compression have sustained a weight of over 300 pounds per square inch without crushing, and when they gave way yielded like rock, and showed no tendency to bulge out at the sides. Water has little or no effect upon this clay, even where the current is extremely strong, but when exposed to the dry air the clay slakes rapidly and crumbles to pieces."

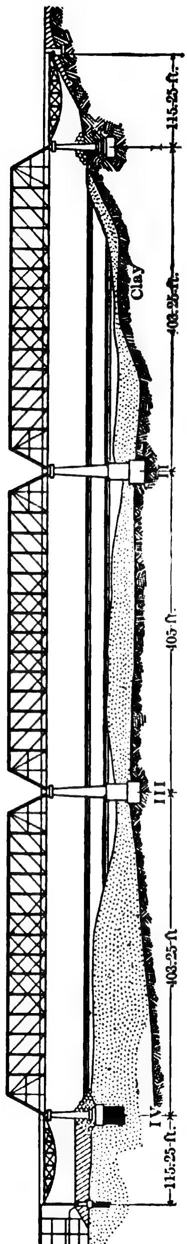


FIG. 161.—BISMARCK BRIDGE FOUNDATIONS.

The Roebling Suspension bridge at Cincinnati, Ohio, has the towers (Fig. 162) founded on coarse gravel, although by going 12 feet deeper solid limestone could have been reached. The load of 3.63 tons per square foot adopted was, however, well within safe limits for so large a spread as the base of the towers.

"No definite plan of foundation had been fixed upon; none could be safely adopted before the excavations had been sunk some considerable depth. Whether a solid rock bottom could be reached on either side was uncertain; so also was the question of piling or solid layers of timber left open. It is known that the bed of the Ohio River is throughout its extent underlaid with rock, at no great distance below the surface. But the depths at which the rock is found varies very considerably. At all the rapids and shallows the rock forms the bed itself; in the pools between, heavy layers of gravel, sand, or mud are deposited. At or near the mouths of tributaries, the rock has generally been excavated by the action of the water

to a great depth; the soft material has taken its place, and consequently good foundations can only be made by heavy expenditures. At the site of the bridge, a bed of blue limestone and shale, not very solid, underlies the river bed at a depth of about 12 feet below the lowest part of the channel. A short distance above the bridge this shale is laid bare at low water on the Covington side, a part of it extending

FIG. 162.—CINCINNATI SUSPENSION BRIDGE.

half way across. Under the Covington tower, a heavy bed of coarse sand, mixed with gravel, is found above the rock, while the surface layer is composed of the original clay bottom which forms the river banks. On the Cincinnati side, the original clay or loamy bottom has, to some extent, been washed away, and latterly been filled up again by the materials obtained from cellar excavations.

"In this artificial bank the excavation of the Cincinnati tower was commenced about the 1st of September, 1856, and sunk down to the level of the river, which, during this and the next two months, fell to low-water mark. A little rise of 4 feet intervened, but the river fell again, and continued low until the month of December. It was owing to this remarkably favorable state of the river that we so well succeeded in our foundations, and at a cost which must be considered as very small, considering their magnitude and the sudden floods which may occur at almost any time and sweep over and destroy costly preparations.

"By a wise resolution of the Board of Managers, the work was not to be commenced before a bona fide cash subscription of $300,000 had been secured. Contrary to my expectations, this subscription was rapidly obtained; and in view of the promising state of the river, it was concluded to forthwith commence the foundation work. But no preparations whatever had been made, no materials on hand, no machinery, and no efficient pumps. The total want of the latter proved a very serious drawback, and seriously threatened to defeat the enterprise at the very outset. It is true, the city was full of steamboat-pumps, but of such small dimensions and such construction that they were of no account in such an operation. Raising clean water is an easy process, but to raise large masses of soft mud and sand is not so easy. After experimenting and losing a few precious weeks in an endeavor to work some patent rotary pumps, which utterly failed, we came very near to a complete halt. There was no time to get proper steam-pumps of large dimensions constructed at any of the shops in Cincinnati, nor could we expect them in time from the East; every day's loss was irreparable—and so we were thrown back upon our own resources. Accordingly, I had four large square box-pumps constructed of 3-inch pine plank, strongly hooped, and 24 feet long; one pair 18 inches in the clear, the other 20 inches. Cast-iron gratings with large india-rubber flap-valves formed the piston and lower check-valves; heavy piston-rods, connected with chains which passed over sheaves, and were shackled to rods, extending over the coffer-dam down to the river. These pumps were put up vertically, in two pairs, one pair worked at a time. They were propelled by one of the engines of the then *Champion No. 1*, a powerful towboat, owned by Amos Shinkle, Esq., who generously placed it at my disposal. These pumps worked well and never failed; they threw mud and sand as effectively as pure water, and discharged 40 gallons at each lift.

"When the Cincinnati excavation was commenced, a strong

oak sheet-piling was driven along the river-front to guard against the pressure of water. This, together with a solid embankment, proved a most efficient coffer-dam on the river side. Owing to its low stage, the river gave us no trouble at all. But by the great depth and extent of the foundation, most of the wells along the rising ground, back of Cincinnati, were laid dry. We drained their supplies, and had to pump them out; and this copious influx came from a quarter totally unexpected. The excavation, however, proceeded rapidly day and night, until all the clay and sand was removed, and a deep layer of coarse sand and gravel was laid bare. Soundings were now made by driving long iron bars to the limestone shale underneath, which proved to be about 12 feet lower. A depth of over 12 feet below extreme low water was reached, and the question now arose, whether to go to the rock, to pile, or to lay down a solid timber platform.

"A compact bed of gravel, if left undisturbed, and protected against undermining and washing, stands next to a solid rock foundation, provided that unequal settling is guarded against. Had this tower been located inside of low-water mark, I should have decided upon going down to the rock, although one season's loss would have been the consequence. Piling I considered inferior to the plan adopted, to say nothing of the loss of time. It was therefore decided to stop at the gravel, and to build a solid timber foundation up to low-water mark, thence to commence the masonry. If the timber could be obtained in time in sufficient quantity, the success of this kind of foundation was much more certain to be achieved, and with less risk and cost, than any other plan.

"The timber foundation, thus laid, forms a platform of 110 feet long by 75 feet wide, composed of twelve courses or layers next to the river, but stepped off towards the land side to eight courses, in consequence of the greater hardness of the gravel, almost equal to hardpan. We were obliged to employ all kinds of timber, soft and hard, mixed, as white pine, oak, maple, hickory, button-wood, elm, beech. The length of logs also varied from 25 to 40 feet. They were all flattened and counter-hewed to an even thickness of 12 inches, leaving the other two sides rough. The courses were crossed at right angles; each stick was thoroughly secured by iron rag-bolts 18 inches long and 1 inch in diameter. The vertical joints were left open and filled with clean gravel and broken stone. Every course was leveled off with the adze and then thoroughly grouted with cement grout before the next course was laid down. Care was also taken in breaking the longitudinal joints efficiently. A solid platform of

timber, 110 feet long, 75 feet wide, 12 feet deep on the river side, and 8 feet on the land side, well put together in the manner described, offers a foundation nearly as good as rock, provided it is guarded against undermining.

"The result has fully justified my expectation. I have not been able to discover any settlement during the progress of the masonry; its condition to-day proves the excellence of the foundation. There are 16,000 perches of 25 cubic feet, equal to 400,000 cubic feet, of solid masonry in each tower. Allowing 150 pounds as the average weight of one cubic foot, the total weight of one tower is 60 millions of pounds, or 30,000 tons net. The area of the timber foundation being $110 \times 75 = 8250$ superficial feet, the weight upon each foot is 3.63 tons or 7272 pounds, or $50\frac{1}{2}$ pounds per superficial inch. This is equivalent to a solid mass of iron of 15 feet depth. Now experience proves that such a weight of iron will be supported upon a clay floor, if its surface is well consolidated by tamping. In the case of high chimney-stacks, elevated 300 to 400 feet, a still greater pressure is sometimes produced upon each superficial foot."

The great Brooklyn bridge, also constructed under John A. Roebling, had the towers landed on a few feet of sand overlying bed-rock, and the load was allowed to run up to $5\frac{1}{2}$ tons per square foot.

The bridges in London, England, are nearly all founded upon the stratum known as "London Clay," and the Charing Cross bridge causes a pressure upon it of in the neighborhood of 9 tons per square foot, while on the Cannon Street bridge the load runs to in the neighborhood of $6\frac{1}{2}$ tons per square foot. Both of these structures, however, have shown considerable settlement, and when the great Tower bridge was designed it was decided to reduce these loadings very materially. Tests made by sinking a trial cylinder showed settlement under $6\frac{1}{2}$ tons per square foot, and, disregarding skin friction of the caisson and the buoyancy of the water, 4 tons was adopted as the safe load; although, taking these into account, the actual load per square foot was between 1 and 2 tons.

The cantilever bridge at Memphis, Tenn. (Fig. 163), constructed by George S. Morison, has piers founded upon pneumatic caissons, and tests were made to determine the maximum bearing capacity of the soil, which was a compact clay, and it was found to have an ultimate bearing capacity of $9\frac{1}{2}$ tons.

Although the foregoing represent what is the best practice in regard to the allowable loads per square foot on various classes of soil, they have in many cases been much exceeded. For example,

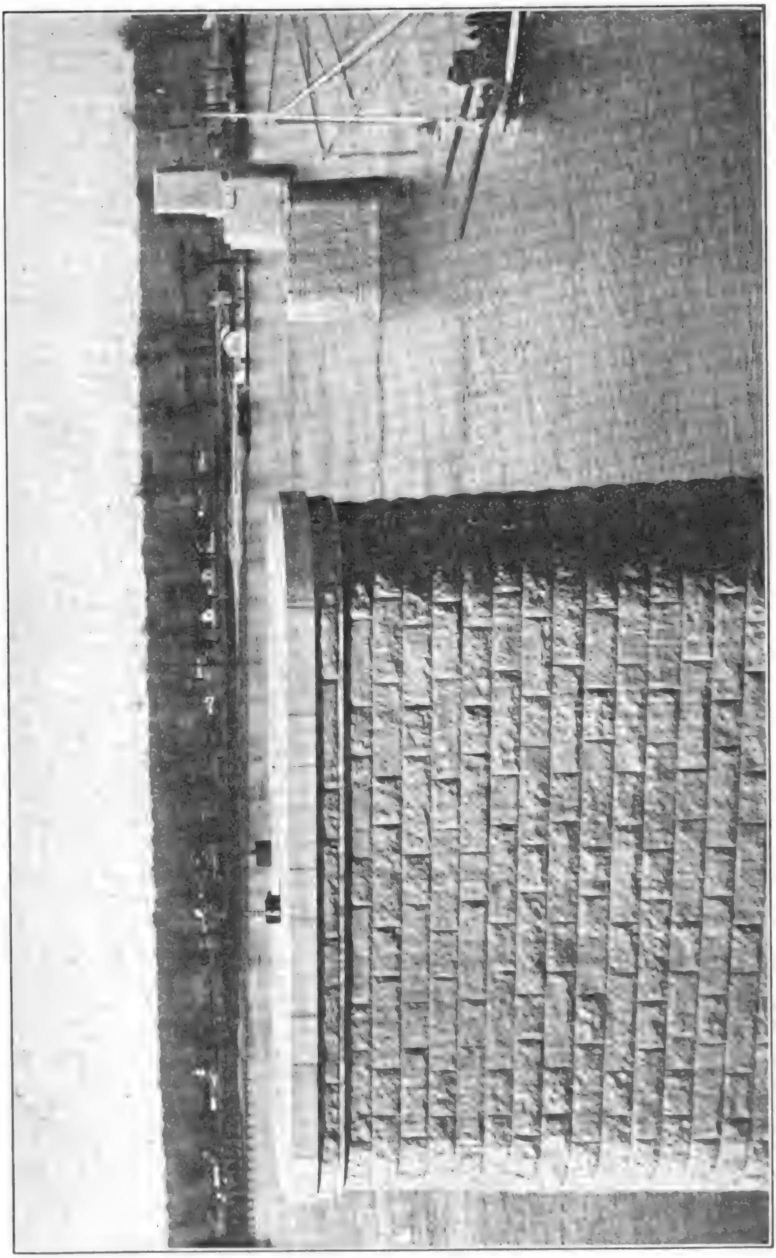


Fig. 163.—Piers of Memphis Cantilever.

the foundation of the Washington Monument causes a pressure on the very fine sand of 11 tons per square foot, although a maximum of about 14 tons is reached during a high wind. Similar overloaded conditions exist with many bridges, as the Gorai bridge causes a pressure of 9 tons on the close-sand foundation, and nearly as much pressure is put upon the sand bottom in the Nantes bridge, where pressure reaches $8\frac{1}{2}$ tons per square foot, although this has settled some, indicating that such high figures are nearer the ultimate bearing capacity than safe loads. A mixture of clay and sand on which the Szegedin bridge in Hungary is founded carries a load of $7\frac{1}{2}$ tons per square foot, although it was found necessary to relieve this foundation by driving piles.

The above data as to the carrying capacity of soil of various kinds may be supplemented by stating that the safe loads for soils per square foot may be rapidly increased for hardpan, cemented gravel, and, of course, very largely increased for rocky ground, as in the case of the Roquefavour aqueduct in France, where then pressure reaches 15 tons per square foot.

Probably the most generally accepted values for foundation loads, that is, the amount which can be placed with safety upon a square foot of foundation bed, are those given by Prof. I. O. Baker in an article published in the *American Architect*, in which he states the maximum allowable load to be 25 tons per square foot for rock of similar hardness as is used in the best ashlar masonry, 15 tons per square foot for rock equal to the best brick masonry, 5 tons per square foot for rock equal to poor brick masonry, 4 tons for dry clay, 2 tons for moderately dry clay, 1 ton for soft clay, 8 tons for cemented gravel and coarse sand, 4 tons for compact and well-cemented sand, 2 tons for clean, dry sand, and 0.5 ton for quicksand and alluvial soils, although these figures can be increased from 25 to 100 per cent., depending upon circumstances and as the judgment of the engineer on the work may dictate.

The building laws of Greater New York are more generally used as a model and authority than any other building rules, and the following extracts cover the provisions as to foundations:

"Where no test of the sustaining power of the soil is made, different soils, excluding mud, at the bottom of the footings, shall be deemed to safely sustain the following loads to the superficial foot, namely:

"Soft clay, 1 ton per square foot;

"Ordinary clay and sand together, in layers, wet and springy, 2 tons per square foot;

"Loam, clay, or fine sand, firm and dry, 3 tons per square foot;

"Very firm, coarse sand, stiff gravel, or hard clay, 4 tons per square foot, or as otherwise determined by the Commissioner of Buildings having jurisdiction.

"Where a test is made of the sustaining power of the soil the Commissioner of Buildings shall be notified so that he may be present in person or by representative. The record of the test shall be filed in the Department of Buildings.

"When a doubt arises as to the safe sustaining power of the earth upon which a building is to be erected the Department of Buildings may order borings to be made, or direct the sustaining power of the soil to be tested by and at the expense of the owner of the proposed building.

"The loads exerting pressure under the footings of foundations in buildings more than three stories in height are to be computed as follows:

"For warehouses and factories they are to be the full dead load and the full live load established by this code, which also gives the loads from other buildings.

"Footings shall be so designed that the loads will be as nearly uniform as possible and not in excess of the safe bearing capacity of the soil, as hereinbefore given.

"Every building, except buildings erected upon solid rock or buildings erected upon wharves and piers on the water-front, shall have foundations of brick, stone, iron, steel, or concrete laid not less than 4 feet below the surface of the earth, on the solid ground or level surface of work, or upon piles or ranging timbers when solid earth or rock is not found.

"Piles intended to sustain a wall, pier, or post shall be spaced not more than 36 nor less than 20 inches on centers, and they shall be driven to a solid bearing if practicable to do so, and the number of such piles shall be sufficient to support the superstructure proposed.

"No pile shall be used of less dimensions than 5 inches at the small end and 10 inches at the butt for short piles, or piles 20 feet or less in length, and 20 inches at the butt for long piles, or piles more than 20 feet in length.

"No pile shall be weighted with a load exceeding 40,000 pounds.

"When a pile is not driven to refusal, its safe sustaining power shall be determined by the following formula: Twice the weight of the hammer in tons multiplied by the height of the fall in feet divided by least penetration of pile under the last blow in inches

plus one. The Commissioner of Buildings shall be notified of the time when such test piles will be driven, that he may be present in person or by representative.

"The tops of all piles shall be cut off below the lowest water-line.

"When required, concrete shall be rammed down in the interspaces between the heads of the piles to a depth and thickness of not less than 12 inches and for 1 foot in width outside of the piles.

"Where ranging and capping timbers are laid on piles for foundations, they shall be of hard wood not less than 6 inches thick and properly joined together, and their tops laid below the lowest water-line.

"Where metal is incorporated in or forms part of a foundation it shall be thoroughly protected from rust by paint, asphaltum, concrete, or by such materials and in such manner as may be approved by the Commissioner of Buildings.

"When footings of iron or steel for columns are placed below the water-level, they shall be similarly coated, or inclosed in concrete, for preservation against rust.

"When foundations are carried down through earth by piers of stone, brick, or concrete in caissons, the loads on same shall be not more than—

"Fifteen tons to the square foot when carried down to rock;

"Ten tons to the square foot when carried down to firm gravel or hard clay;

"Eight tons to the square foot in open caissons or sheet-pile trenches when carried down to rock.

"Wood piles may be used for the foundations under frame buildings built over the water or on salt-meadow land, in which case the piles may project above the water a sufficient height to raise the building above high tide, and the building may be placed directly thereon without other foundation.

"Foundation walls shall be construed to include all walls and piers built below the curb level, or nearest tier of beams to the curb, to serve as supports for walls, piers, columns, girders, posts, or beams.

"Foundation walls shall be built of stone, brick, Portland cement concrete, iron, or steel.

"If built of rubble stone or Portland cement concrete, they shall be at least 8 inches thicker than the wall next above them to a depth of 12 feet below the curb level; and for every additional 10 feet, or part thereof, deeper they shall be increased 4 inches in thickness.

"If built of brick, they shall be at least 4 inches thicker than the wall next above them to a depth of 12 feet below the curb level;

and for every additional 10 feet, or part thereof, deeper they shall be increased 4 inches in thickness.

"The footing or base course shall be of stone or concrete, or both, or of concrete and stepped-up brickwork, of sufficient thickness and area to safely bear the weight to be imposed thereon.

"If the footing or base course be of concrete, the concrete shall not be less than 12 inches.

"If of stone, the stones shall not be less than 2×3 feet, and at least 8 inches in thickness for walls; and not less than 10 inches in thickness if under piers, columns, or posts.

"The footing or base course, whether formed of concrete or stone, shall be at least 12 inches wider than the bottom width of walls, and at least 12 inches wider on all sides than the bottom width of said piers, columns, or posts.

"If the superimposed load is such as to cause undue transverse strain on a footing projecting 12 inches, the thickness of such footing is to be increased so as to carry the load with safety.

"For small structures and for small piers sustaining light loads, the Commissioner of Buildings having jurisdiction may, in his discretion, allow a reduction in the thickness and projection for footings or base courses herein specified.

"All base stones shall be well bedded and laid crosswise, edge to edge.

"If stepped-up footings of brick are used, in place of stone, above the concrete, the offsets, if laid in single courses, shall each not exceed 1½ inches, or if laid in double courses, then each shall not exceed 3 inches, offsetting the first course of brickwork, back one-half the thickness of the concrete base, so as to properly distribute the load to be imposed thereon.

"If, in place of a continuous foundation wall, isolated piers are to be built to support the superstructure, where the nature of the ground and the character of the building make it necessary, in the opinion of the Commissioner of Buildings having jurisdiction, inverted arches resting on a proper bed of concrete, both designed to transmit with safety the superimposed loads, shall be turned between the piers. The thrust of the outer piers shall be taken up by suitable wrought-iron or steel rods and plates.

"Grillage beams of wrought iron or steel resting on a proper concrete bed may be used. Such beams must be provided with separators and bolts inclosed and filled solid between with concrete, and of such sizes and so arranged as to transmit with safety the superimposed loads.

"All stone walls 24 inches or less in thickness shall have at least one header extending through the wall in every 3 feet in height from the bottom of the wall, and in every 3 feet in length, and if over 24 inches in thickness, shall have one header for every 6 superficial feet on both sides of the wall, laid on top of each other to bond together, and running into the wall at least 2 feet.

"All headers shall be at least 12 inches in width and 8 inches in thickness and consist of good flat stones.

"No stone shall be laid in such walls in any other position than on its natural bed.

"No stone shall be used that does not bond or extend into the wall at least 6 inches.

"Stones shall be firmly bedded in cement mortar and all spaces and joints thoroughly filled."

The subject of Allowable Pressure on Deep Foundations is discussed in a valuable monograph by the eminent engineer, Elmer L. Corthell. As Consulting Engineer for the port works of the Argentine Government, it became necessary to arrive at a reliable figure for the pressure proper to allow on the bottom for sea-walls to be founded by compressed air. Finding the data to be very unsatisfactory, Mr. Corthell caused a search to be made, and has published in the monograph mentioned the results of his investigations. As every engineer engaged in foundation work should own a copy of this book, only the conclusions drawn by Mr. Corthell will be given:

"This analysis is based on the various classes of material so far as they could be ascertained and classified.

"The pressures of stable structures on fine sand range from 2.25 tons of 2000 pounds to 5.80 tons, with an average of 4.5 tons with ten examples.

"On coarse sand and gravel from 2.40 tons to 7.75 tons, with an average of 5.1 tons with thirty-three examples.

"On sand and clay from 2.5 tons to 8.5 tons, with an average of 4.9 tons with ten examples.

"On alluvium and silt from 1.5 to 6.2 tons, with an average of 2.9 tons with seven examples.

"On hard clay from 2.0 tons to 8.0 tons, with an average of 5.08 tons with sixteen examples.

"On hardpan from 3.0 tons to 12.0 tons, with an average of 8.7 tons with five examples.

"The above cases show no settlement. The range is considerable, and, no doubt, in the case of the minimum pressure a much larger weight could have been imposed on the material without producing

settlement. So that, for a safe rule, the average is low and a safe one would lie somewhere between the averages above given and the maximum pressures.

"We find three cases where notable settlement took place in fine sand where the range was from 1.8 ton to 7.0 tons, and the average was 5.2 tons; no doubt the case of the minimum was one of loose quicksand unconstrained.

"In clay—largely cases of London clay—we find five examples where the pressures range from 4.50 tons to 5.60 tons, with an average of 5.2 tons, quite uniform pressures.

"In silt and alluvium we have two cases of settlement which were 1.6 ton and 7.6 tons, a wide variation.

"There are three cases of failure on sand and clay mixed, the pressures ranging from 1.6 tons (Chicago) to 7.4 tons, an average of 3.3 tons. It is to be noted that there was given above an average of 4.9 tons, and ten examples, ranging from 2.5 tons to 8.5 tons, where no settlement ocurred in similar material.

"The records of frictional resistance are quite variable also. In ten cases of cylinder piers, the average was 540 pounds per square foot, ranging from 300 to 1500 pounds, gravel appearing to show the greatest amount (1500 pounds), and mud the least.

"In respect to masonry piers, of which we have twenty-three examples, the range is from 300 pounds per square foot in sand and gravel to 1000 pounds in sand and clay, with an average of 522 pounds. Walls, quays and otherwise, show an average of 270 pounds per square foot, with a range from 205 pounds to 450 pounds with five examples."

Some very valuable data on the bearing capacity of clay is given in Volume II on the Antwerp quay wall, and a very valuable formula given for varying depths. A general formula will be found in Chapter XIV, based on the Antwerp formula.

CHAPTER XII

LOCATION AND DESIGN OF PIERS

PIERS of a bridge cannot always be located with reference to easy construction nor spaced at economical distances apart. In thickly settled parts of a country, or as part of an existing line of communication, the bridge must be located usually in a position previously determined, and the piers can only be spaced with regard to economy, provided due regard can at the same time be paid to the needs of navigation, government requirements, and sufficient waterway.

Where the bridge is to be constructed in a new country, or upon a new line of road, the crossing should be selected where the river is of moderate width; that is, not so wide as to demand a structure of excessive length and probably of excessive cost, nor so narrow that the current will be exceedingly swift and make the foundations very difficult and costly to build, unless, of course, it is narrow enough to admit of using a one-span structure at a reasonable cost. The two-hinged plate-girder arch, Fig. 164, constructed by the author at Youngstown, Ohio, has a span of 210 feet, and while constructed largely for æsthetic reasons, it was also economical on account of the very considerable height, to avoid a pier in the river.

On all the large navigable rivers, the channel is fixed and the length of the channel span prescribed by law, as is also the method of procedure in obtaining the approval of the government engineers. The Secretary of War must be furnished with a copy of the State law authorizing the construction of the bridge, certified to by the Secretary of State under seal; drawings in triplicate showing the general plan of the bridge; a map in triplicate showing the location of the bridge, giving, for the distance of one mile above and one-half mile below the proposed location, the high- and low-water lines upon the banks of the stream, the direction and strength of the current at high and low water, with the soundings accurately showing the bed of the stream, and the location of any other bridge or bridges, such map to be sufficiently in detail to enable the Secretary

Fig. 164.—Plate Girder Arch, Youngstown, Ohio.

of War to judge of the proper location of the bridge. In addition to the above, if the applicant is a corporation, there will be required a certified copy of its articles of incorporation, a certified copy of the minutes of the organization of the company, and an abstract of the minutes of the corporation, showing the present officers of the company, all duly certified to.

When the location of the bridge has been made, a thorough examination of the site must be instituted. Soundings must be made to determine the depth of the stream at low water. Ordinary and extreme high-water lines must be established and the flow of the stream be obtained. A careful examination must be carried out as to the character of the river bed, and drillings made to learn the character and thickness of strata and the distance to bed-rock, as well as the quality of it.

Borings to a small depth may be made by hand-drills (Fig. 165, *A*), which are operated by striking with a sledge and turned constantly

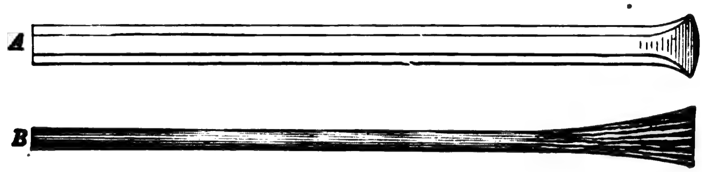


FIG. 165.—HAND-DRILL AND SWAB.

to keep a round bore, or if long and heavy they will cut their way, if simply raised up and allowed to drop, with their own weight. The hole is kept partly filled with water and can be cleaned out with a small sand-pump or with a swab (Fig. 165, *B*) made from a stick slivered at the end, which will also bring up samples.

The Pierce steel prospecting auger is a tool which can also be used without a derrick to bore test holes from 10 to 50 feet into loose soils or clay. Holes from $2\frac{1}{2}$ to 6 inches in diameter can be drilled and samples obtained. The auger can be turned either by hand-wrenches or by horse-power.

Where the borings are to be of an extensive character a well-drilling machine can be utilized, such as shown in Fig. 166, and which can be run onto an ordinary flatboat and towed to place.

The tools for drilling are a temper screw for regulating the height of the drill, a sinker bar to give the weight, steel jars, and drilling-bits. A sand-pump is used to clean the hole and obtain samples; rope-spears, rope-knives, and fishing-tools to remove lost rope, tools, and pebbles or other obstructions. The drill holes, unless

through rock, are cased with iron pipe which can be withdrawn when the hole is completed.

The borings made by the Mississippi River Commission were

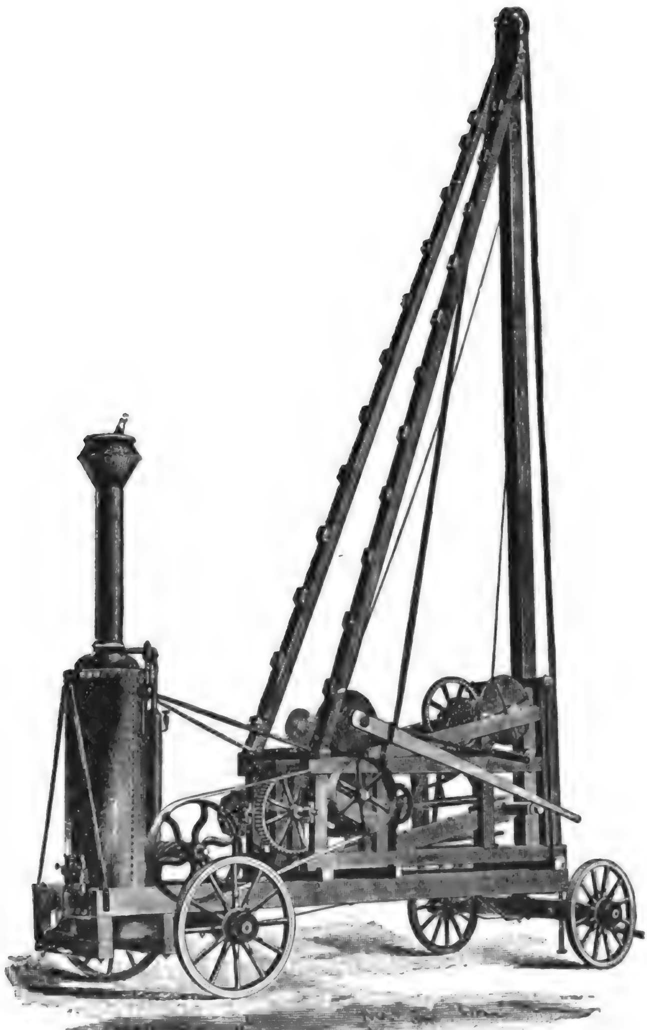


FIG. 166.—STEAM-POWER WELL-DRILLER.

very extensive, and a special tripod apparatus (Fig. 167) was devised with a view to easy transportation and easy repair in the field. The tripod was 30 feet in height, with a strong head or cap, surmounted

by a galvanized-iron guide-pipe 20 feet in height, in two sections, and held in place with guy-ropes. The men operating the tools stood upon the triangular platforms which were attached to the

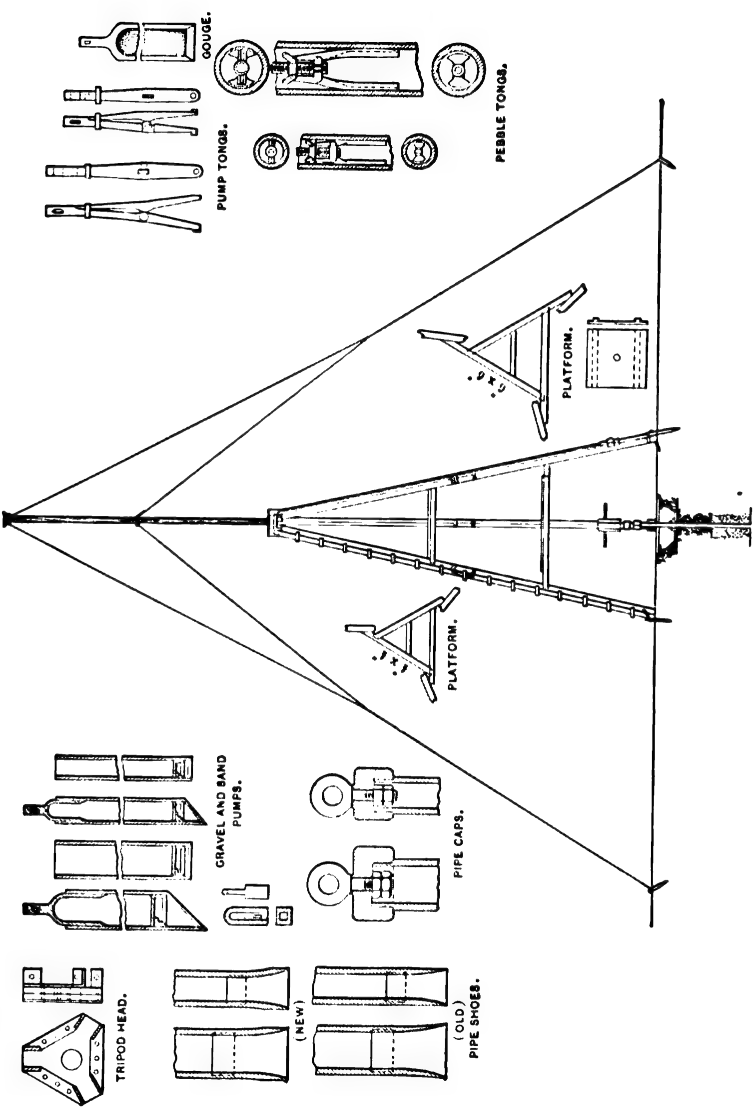


FIG. 167.—TEST-BORING APPARATUS, MISSISSIPPI RIVER COMMISSION.

legs. The casing was iron pipe in 10-foot lengths and screwed together so as to present a smooth surface, while the bottom was provided with a steel cutting-shoe, having a mouth slightly larger than the pipe.

The sinking is accomplished by driving and twisting, the driving being done by means of the clamp on the pipe and the maul sliding on the pipe. (Fig. 168). The weight of the maul is from 80 to 100 pounds and is worked by three men giving it a lift of about 2 feet, the best results being obtained when the men act in concert and give rapid blows. The removal of the core and samples is accomplished by means of the various tools shown in Fig. 315, and requires great care and considerable experience. The pump was raised and lowered by means of the reel attached to one leg of the tripod, and its distance from the surface noted from graduations on the pump-rod. When the boring is completed the tube is withdrawn by a system of compound levers, assisted by a set of differential blocks when necessary, as the force exerted was often as much as the strength of the pipe at the joints. The pebble-tongs were for use in removing large pebbles which would not enter the pumps, and for recovering lost tools or the pump itself in case of becoming detached.

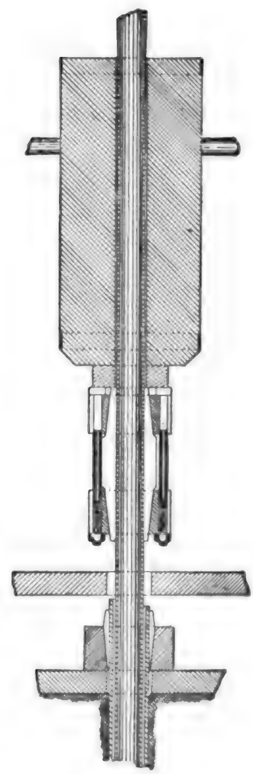


FIG. 168.—CLAMP AND MAUL.

The above account is taken from the report of J. W. Nier, assistant engineer, to which reference must be made for other details.

The poor results obtained in examining the bottom by means of any of the preceding methods of making borings has been mentioned in a number of places in the preceding chapters, and practically the only type of borings that can be relied upon are core borings, made with a diamond drill, or some modification of it.

The borings for the Chicago & Northwestern Railroad Company bridge over the Mississippi River at Clinton, Iowa, were first made by a churn drill, as shown in Fig. 169, where the dotted line was the supposed bed-rock. By the use of a Sullivan diamond drill the bed-rock as shown by the solid line was actually found in some places to be over 30 feet below the original rock profile.

Core borings can be made with a diamond drill, operated by gasoline or steam, Fig. 170, or by hand, Fig. 171. The diamond drill consists of a hollow bit in which are set " black diamonds " or carbon. This

bit is attached to a hollow rod made up in 5- or 10-foot sections, screwed together so that the tool can be lengthened as the depth of the hole increases. About every 8 or 10 feet the drill rod is hoisted out and the core removed, it being caught or held by a core-lifter. This apparatus will operate just as well through the water or soft material as it would to start in on the dry ground surface, so that for testing the bed of a river or the bottom under any body of water, it can be readily used, and will give the exact truth. It is necessary that great care should be exercised in selecting the carbon best suited for this work, and this should be done by the manufacturer or someone having considerable experience. Where the work is not

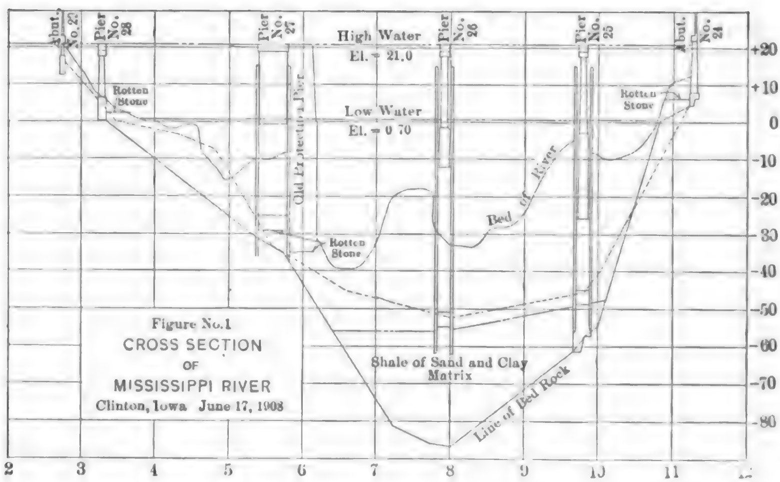


FIG. 169.—TEST-BORING BY DIAMOND DRILLS. C. & N. W. RY.

extensive enough to break in a crew of men, they should be obtained through someone that can furnish at least an experienced operator, although where the work will last some time, a first-class stationary engineer can soon learn to detect when the drill is getting into loose material that will cave, and so avoid trouble. An unskillful operator will often lose the hole or cause a great amount of wear on the tools and break up a lot of carbon. The diamonds, or carbon, must be set in the bit by someone having experience in fitting them up. Work of this character will be undertaken on contract by the manufacturers at varying prices, according to the amount and character of the work.

The cost of the drilling for the Chicago & Northwestern R. R. work already referred to averaged $1.83 per vertical foot. These

cores were 2 inches in diameter, although smaller cores can be taken out if the work is done carefully. Where there is a very extended series of borings to be made, the cost might be reduced to as low as $1.50 per foot, but will range from this to $2.50 to $3 per foot

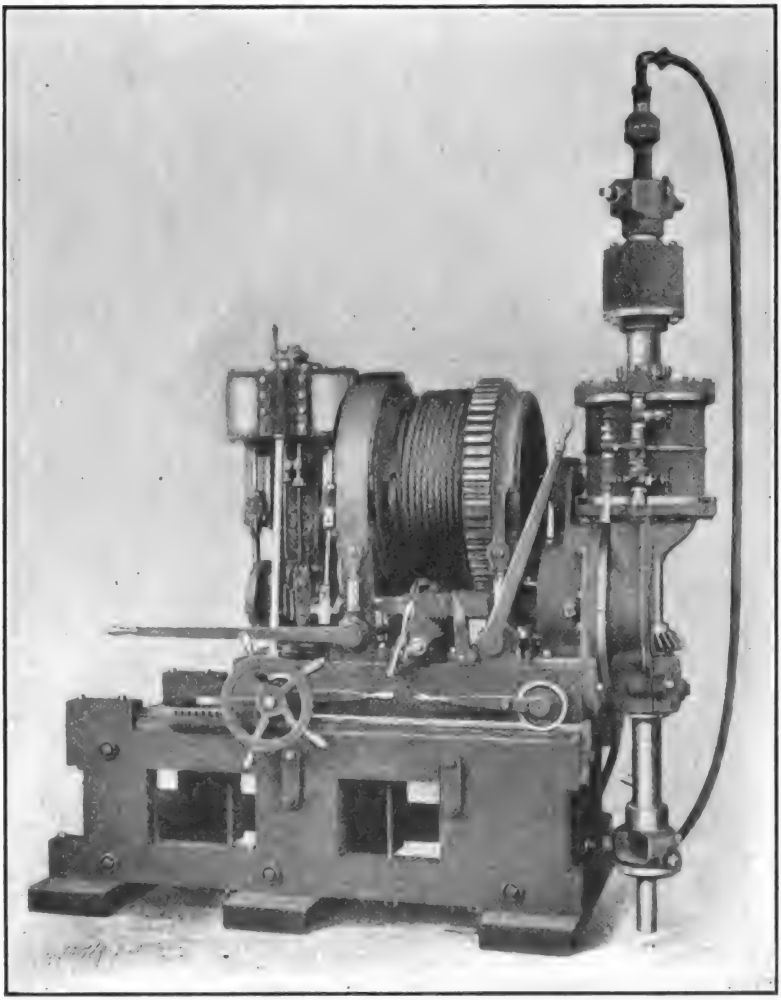


FIG. 170.—SULLIVAN POWER DIAMOND DRILL.

for difficult work; but this is low-priced insurance for knowing exactly what class of material is being dealt with.

The conclusions arrived at by F. H. Bainbridge, Engineer on the Chicago & Northwestern R. R., are as follows:

"The final location of the caisson can be accurately determined by core borings, and cut stone and timber ordered without any waste or delay waiting for material for which no provision had been made.

"The contractor in bidding on the work knows exactly what material is to be encountered, and will make a lower bid when there is no uncertainty. The difference in cost between handling in a caisson, material which can be taken out through the blowpipe, and material which must be locked out in buckets is very great.

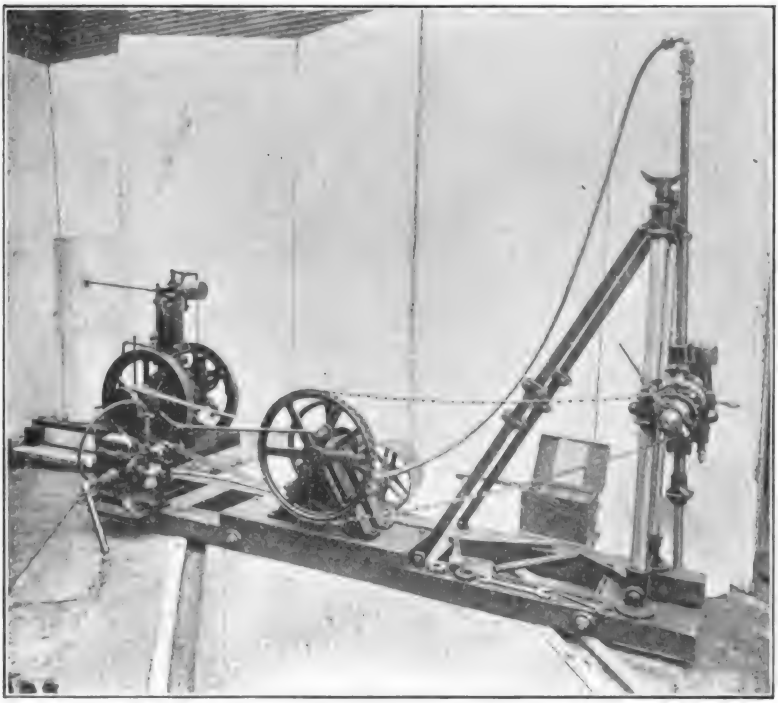


FIG. 171.—SULLIVAN HAND AND POWER DIAMOND DRILL.

"The piers can be located in the most economical position. Often a change of a few feet in locating a pier may make a difference in cost of tens of thousands of dollars.

"Much can be learned as to the character of the foundation that cannot be learned from the interior of the caisson. In limestone formations subterranean caverns are common, and in both lime and sandstone formations overhanging subterranean cliffs are found. The existence of these can be determined with the drill, but cannot be learned from the interior of the caisson."

The McKiernan-Terry core drill is operated with a shot bit instead of a bit set with diamonds, and many records show it to be less expensive for making borings than the diamond drill, cores hav-

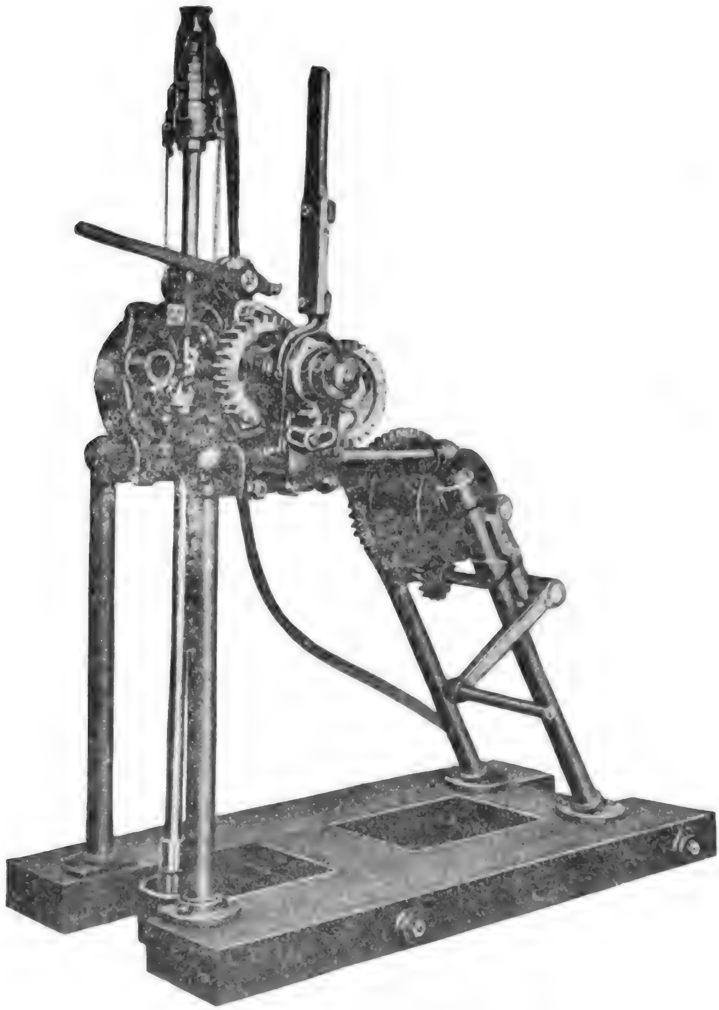


FIG. 172.—MCKIERNAN-TERRY CORE DRILL.

ing been taken out up to 4 inches in diameter at a total cost of less than $1.50 per foot. It is, of course, possible to get better cores from friable material with a large core than with a small one, and perfect cores up to $16\frac{3}{4}$ inches are taken out with the standard machines,

but they may be fitted with tools up to 60 inches in diameter where it is only necessary to go but a short distance. The class "Z-1" drill is shown in Fig. 172, and the shot bit is shown in Fig. 173. The drilling, of course, is not done with perfect shot, but by the broken pieces which are angular in shape.

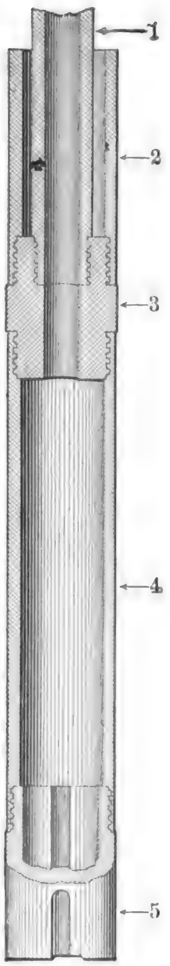


FIG. 173.—SHOT BIT FOR CORE DRILL.

The class "Z-1" drill is employed for prospecting and testing work. Its light weight commends its use for such purposes, particularly in places inaccessible to a larger and heavier apparatus. The net weight of the drill shown in the cut on the preceding page is 437 pounds. It can be knocked down and boxed in packages of convenient size for mule-back transportation.

The capacity of the "Z-1" drill is 400 feet, to which depth it will bore a $2\frac{1}{4}$-inch hole and cut a $1\frac{1}{4}$-inch core. Larger tools may be used, which will bore a 3-inch hole to a depth of 250 feet and cut a $1\frac{3}{4}$-inch core. The power to operate this drill is generally supplied by a gasoline motor, to which it is belt-connected, although a steam engine, electric motor, or horse-power may be used if preferred. The drill is fitted with a swivel head, which permits of drilling up to an angle of 45 degrees with the vertical.

On the rear of the frame is mounted a hand hoist, used for raising or lowering the drill rods in the bore hole and for driving casing through soil to bed-rock. Pressure on the drilling tools is applied by means of the sensitive feeding device shown in the cut.

A tripod derrick is generally used with this drill, and, as a rule, it is built from timber cut on the ground. It should be of sufficient height to admit of two lengths, or 20 feet, of drill rods, being raised at a pull. For shallow holes, where but little power is required to rotate the rods, the drill can be arranged for operation by hand. Particulars regarding such equipment will be furnished on application.

When the examination of the site has been completed and the borings finished, the form of foundations may be decided upon, due weight being given to good foundations and to the allowable expenditure. Should the obtaining of good foundations be seen to

be very expensive, long spans must be adopted to require few piers in the river; but if inexpensive, much shorter spans, with more piers, may be used.

The length of spans for a least cost of structure was formerly assumed to be decided when the cost of one span was made equal to the cost of one pier, and for spans of certain capacity this might be approximately true, but a very neat mathematical solution of this problem by Alfred D. Ottewell, consulting engineer, was published in the *Engineering News* of Dec. 14, 1889. The total length of the structure in feet was represented by l, the number of spans by n, the length of one span in feet $l \div n$ by s, the cost of one span in dollars by c, the cost of one pier in dollars by p, the total cost of the structure in dollars by y, while a and b are constants.

From the estimated cost of a large number of spans, a curve of costs was plotted and the following equation of a parabola deduced:

$$c=a+\frac{(s-20)^2}{b}. \qquad (1)$$

Since there are n spans and $n+1$ piers, the total cost of the structure would be

$$y=nc+(n+1)p. \qquad (2)$$

Then by substituting the value of c from (1), reducing and making the first differential coefficient equal to zero, the cost of one pier is obtained, which will make the total cost of the structure a minimum, or

$$p=\frac{s^2-(ab+400)}{b}. \qquad (3)$$

Or when the cost of a pier has been estimated, the economical length of span may be found by a transposition of the above formula:

$$s=\sqrt{ab+400+pb} \qquad (4)$$

The values of a and b may be found by substituting in equation (1) computed values of the cost of a number of spans for an actual loading. Values of s, p, and c may then be computed and tabulated for spans from 100 feet upwards, as formula (1) is not true for shorter lengths.

In an actual calculation for B. & O. R.R. loading, which consists of two 125-ton engines followed by a 4000-pound per lineal foot train-load, a was found equal to 1950 and b to 3.05. Assuming a case where the length of the bridge is 700 feet, where the height of the piers will average 25 feet, and the average cost of piers and abutments be $4310, then from formula (4) the economical span will be found equal to 160 feet. The total cost of the structure will be found, by using formula (1), and the cost of piers as above, to be $59,700; while with only four spans of 175 feet the total cost would exceed $60,800, and with six spans of 117 feet would be about $61,400.

Should there be any doubt as to the ease of obtaining foundations, the prudent engineer might deem it wise, however, to build the four-span structure and avoid the risk and delay which would be caused by another foundation in the river.

After deciding upon the number and location of the piers, they must be designed with reference both to their being as slight obstructions to the water as possible and to their architectural appearance.

Particular attention was given to the design of piers by the late Geo. S. Morison, consulting engineer, whose work on the bridges across our great rivers is notable for its strength, simplicity, and finished appearance. In a lecture he described the process of the design of some large piers: "Fourteen years ago I had occasion to design a bridge pier for a bridge across one of our Western rivers, and I tried to make an ornamental pier. When the plans were completed I did not like them. One change after another was made, all tending to simplicity. Finally the plans were done. From high water down, the pier was adapted to pass the water with the least disturbance; it had parallel sides and the ends were formed of two circular arcs meeting. Above high water the ends were made semicircular instead of being pointed. The pier was built throughout with a batter of one in twenty-four. A coping 2 feet wider than the body of the pier projected far enough to shed water, and the projection was divided between the coping and the course below. Another coping with a less projection surmounted the pointed ends where the shape was changed. It was as simple a pier as could be built, and in every way fitted to do its duty. I had started to make a handsome pier. The pier that was exactly what was wanted for the work was the only one that satisfied the demands of beauty. Forty-three piers of precisely this design, no change having been made except in the varying dimensions required for different structures, besides eight others in which only the lower parts are modified, are now standing in eleven different bridges across three great Western

rivers. In designing a pier it must be remembered that the portion of the pier below the water has more to do with the free passage of the water than that above water. In a deep river the model form of the pier should begin near the bottom of the river and not at low water. Many rivers in flood time carry a great amount of drift. A pier like that which I have described catches but little of this drift. If, however, a rectangular foundation terminates but little below water, that foundation may both disturb the current and catch the drift."

The piers of the Omaha bridge which carries the Union Pacific across the Missouri River are illustrated in Fig. 174, and were constructed as described and are among the most beautiful piers in this country.

In Europe, where money is more lavishly expended on great works of engineering, piers of great architectural beauty are much more frequent. The Russian Government railways, which have seemingly been constructed without regard to expense, have many beautiful examples of bridge masonry and piers; the view of one of them (Fig. 175), with curved ends, shows the elegant and massive character of the masonry. While extremely simple in design, the cut-stone coping and the molded corbel course below give it a finish which cannot be surpassed.

The design of piers for strength and stability is fully treated in Baker's "Masonry Construction," but some experiments, which were made with reference to the proper form to occasion the least resistance, will be quoted at length from Cresy.

The introduction of piers into a channel gives rise to a great disturbance in the velocity and flow of the water. Rapid currents are formed which cause the bed of the stream to become washed and the foundations to be endangered; eddies are created which are likewise undesirable, and it becomes necessary to adopt such a form for the ends of the piers that the disturbance to the flow shall be small.

M. Bossut, in a French work on jetties, thought to have solved this problem by mathematics, his conclusion being that the starling should be triangular, the nose being a right angle.

M. Dubaut, in his "Principles of Hydraulics," gave another solution which was more nearly the truth, in that he arrived at the conclusion that the faces of the starling should be convex curves. The true form is most nearly reached when these curves are tangent to the sides of the pier, and, further than this, regard must be paid to giving enough solidity to the starlings to protect them from ice

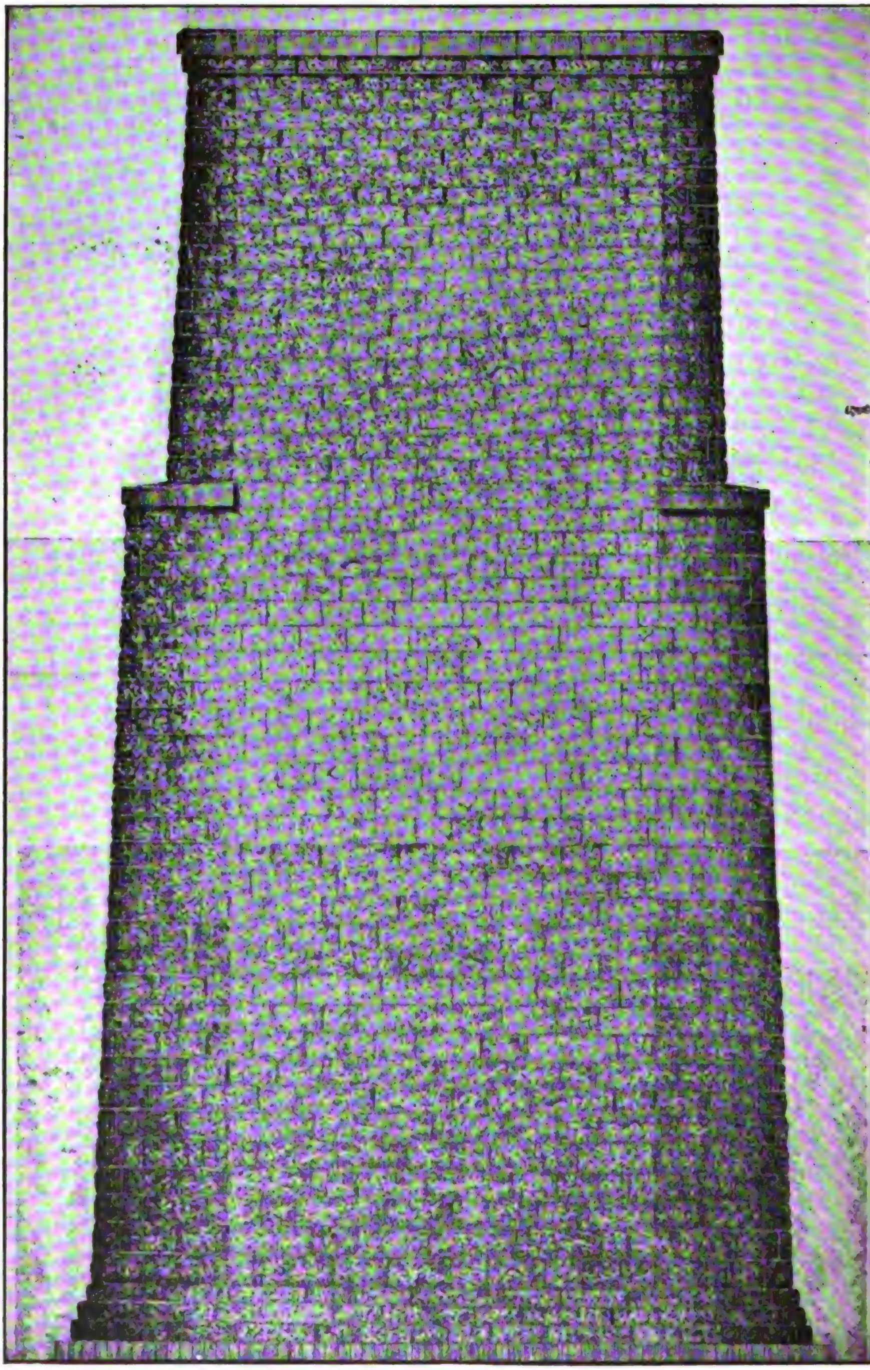


FIG. 174.—PIER OF OMAHA BRIDGE, UNION PACIFIC SYSTEM.

and drift. A happy medium would seem to be reached by making the curves with a radius equal to one-sixth of the circumference, described on the sides of an equilateral triangle.

Experiments were made with models of different forms, which were placed in a rectangular canal between boards of 50 centimeters in length, in which the water flowed about 40 millimeters in height, the models being 15 centimeters in thickness. By means of a fall,

FIG. 175.—RUSSIAN PIER, RUSSIAN STATE RAILWAYS.

the water was given a velocity of 3 meters 9 centimeters per second, the contraction, eddies, and currents being carefully measured. The first experiment was made on a pier (Fig. 176 *a*) with rectangular starling. An eddy was formed before the pier 34 millimeters high, in a nearly circular band A, falling nearly vertical at the corner. There were two other currents along the faces of the pier, the height of which can be seen in the cross-sections.

The second experiment (Fig. 176 *b*) was with a triangular star-

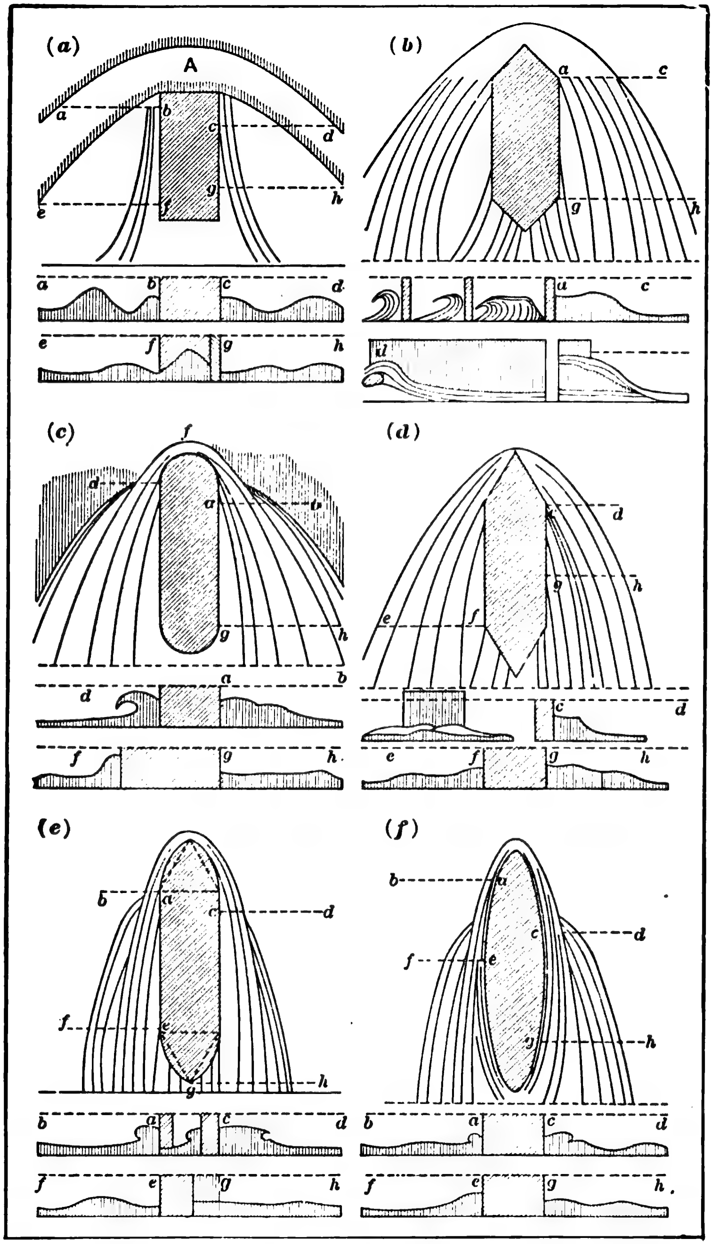


FIG. 176.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

ling, the nose being a right angle. It formed a less obstruction than the square end, but the fall at the shoulder was as deep and more dangerous, while eddies were formed as seen in the sections.

The third one (Fig. 176 *c*) had a semicircular starling. The eddy was not so wide, but nearly as high.

The fourth model had a triangular starling, with an angle of 60° at the nose (Fig. 176 *d*). The eddy was less, as was also the fall at the shoulder.

The starling in the fifth was formed by two circular arcs, tangent to the sides and described on the sides of an equilateral triangle (Fig. 176 *e*). The eddy was small and there was no fall at the shoulder.

The sixth (Fig. 176 *f*) was a model the plan of which was an ellipse, of which the small diameter was one-fourth the length, and the eddy was less than any of the others.

The seventh model (Fig. 177 *a*) had a starling with concave faces, such as is sometimes used where the wing-wall meets an abutment. It produced the most dangerous currents of all.

The eighth (Fig. 177 *b*) was of the same form as Fig. 177 *e*, but the water was supposed to mount the springing of the arch.

The ninth and tenth experiments (Figs. 177 *c* and *d*) were on the same forms as Figs. 176 *e* and 176 *f*, but the current had a velocity of 4 meters 87 centimeters per second, such as a river would have in its overflow. The eddy (Fig. 177 *c*) rose to nearly twice the height, as was the case with the lesser velocity, and, while there was no fall, the inclination formed along the faces was more rapid.

The effect with this velocity on the elliptical pier (Fig. 177 *d*) was similar to the lesser velocity, but more marked. It may thus be concluded that the elliptical section offers the least resistance to the current and occasions the least contraction, while the form with convex starling comes next, and of piers with triangular starlings the one with the 60° nose is the best.

Where ice is to be provided for, the nose is often inclined to allow large cakes to mount it and break in two, without doing further damage. For any large or important structure, the design of the piers should receive a great deal of study, and be designed not only with reference to their theoretical form, but with reference to the form of pier which has shown the best results practically and has been found to be best suited to the velocity of the stream in which they are to be built, and to best withstand the drift and ice that may be met, with giving at the same time all the consideration

possible to the architectural effect and to the harmony with the entire structure.

The piers of the Knoxville steel arched cantilever, Figs. 178–9, were equally spaced by the author in designing the structure, in order to make the bridge symmetrical and a good piece of architecture.

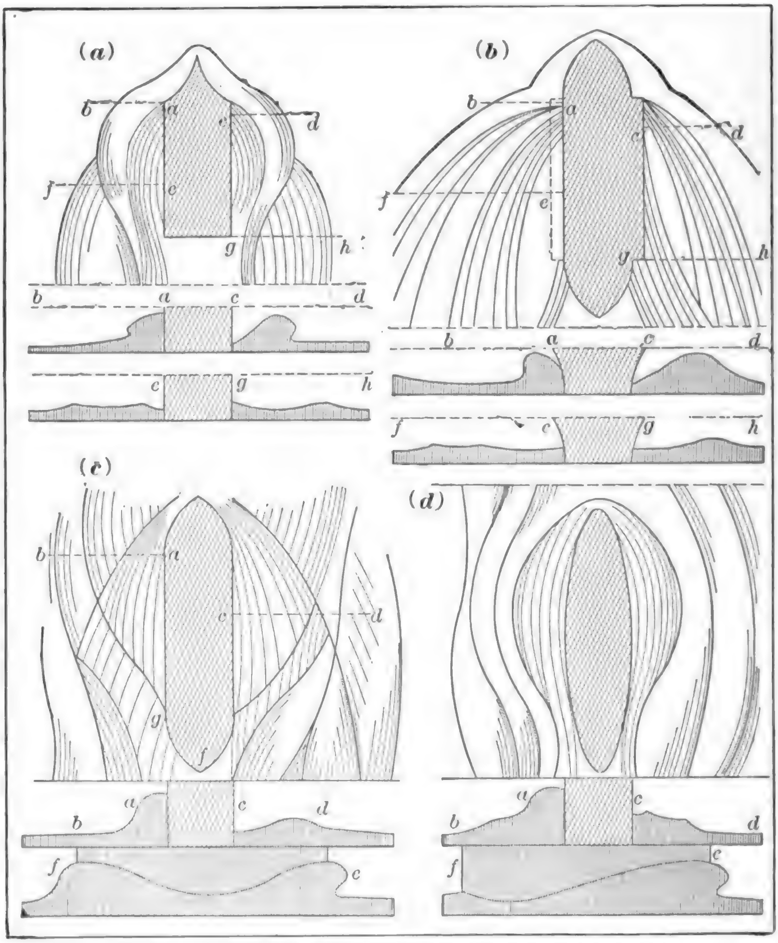


Fig. 177.—Cresy's Experiments on the Form of Piers.

The piers had a cross-section as in Fig. 177 *c*, as the current in the river was very swift during high water. Had more money been available to add ornamental corbels and copings and proper capping around the steel shoes and bolsters, the appearance would have been much improved.

Fig. 178.—Knoxville Steel Arched Cantilever.

Fig. 179.—Knoxville Steel Arched Cantilever During Erection.

CHAPTER XIII

LOCATION AND DESIGN OF PIERS (CONTINUED)

THE stones most used in the building of piers are granite, sandstone, marble, and limestone, the amount of granite used for this purpose being about the same or possibly a little less than sandstone. Next to these, limestone is used most largely and marble the least of the four, owing to the fact that most marble is suitable for dressed stonework for buildings and too expensive for use in piers.

Granite is most largely produced in the New England States, and notably in Maine, Massachusttes, and Vermont. Next to these States comes California, with about as much of an output as Vermont. Only fifteen of the States in the Union do not have an output of granite, so that it may be said to be available at greater or less cost in any part of the United States.

Sandstone is most largely produced in Pennsylvania, Ohio, and New York, although but thirteen of the States are non-producers of this most commonly used building material.

Marble is most largely produced in Vermont, although New York, Tennessee, and Georgia have extensive quarries and are large producers.

Limestone is extensively quarried in Pennsylvania, Ohio, New York, Missouri, Wisconsin, Illinois and Indiana, only six States being without output of this stone.

Granite is the best building stone for use in constructing piers, and under this head are included the true granites, gneiss, mica-schist, andesite, syenite, and quartz-porphyry.

Sandstone covers all the consolidated sands, the strength of sandstone depending entirely upon the cementing material, as it is simply quartz grains cemented together, and when silica is the cementing material it is the very best. Quarrymen term sandstone according to its quality as bluestone, freestone, and conglomerates.

Limestone is usually the least valuable of stone that is used in building piers, owing to the fact that it is too soft and stands the weather poorly. It consists practically of amorphous calcium car-

bonate, sometimes cemented together by crystalline substances with many impurities. Under the head of limestone is also usually included magnesium carbonate, called magnesium limestone, and when the stone is about equally made up of carbonate of lime and magnesium it is termed dolomite. Dolomite is much harder than ordinary limestone, and consequently forms a much better building material. Chemically, there is little difference between limestone and marble, except that marble has been crystallized by the action of heat.

Of the quality of the granite to be obtained in any part of the United States very little need be said, as it is always a desirable building material, and only the question of cost comes up for consideration as to whether it can be used or not. In the New England States, Georgia, California, and Washington it is cheap enough to make it possible to use it wherever a reasonable amount of money is available for the construction of bridge piers or foundations of any sort. The softer sandstones obtainable throughout the country and which have a compressive strength of about 5000 pounds per square inch are suitable for building bridge piers where not too much ice or abrasive action of any character is to be contended with.

The freestones of northeastern Ohio, and similar stone wherever found throughout the United States and similar to the Chuckanut stone of Puget Sound, are almost as desirable as granite for the construction of piers.

Limestone does not very often occur of such quality as to warrant its use in masonry construction, and while it may seem to be of sufficient hardness to warrant consideration, attention need only be called to its use in the State Capitol building of Ohio, where it has weathered badly, to make it seem advisable to find some other material if possible.

Where marble is plenty, it is of course of sufficient strength and hardness to make it desirable for use in foundation work and for piers, provided the cost is not excessive, and a poorer quality of marble found in Tennessee and Georgia, and known as iron limestone, is certainly one of the best building stones to be found anywhere. The most famous limestone to be found in the United States is the oolitic limestone of Indiana. It is very easily quarried and hardens on exposure to the air, so that it is quite durable; and, on account of the large size of the blocks in which it can be gotten out, is very much used for massive work.

One of the things least often considered in the selection of stone for ordinary piers is the color; although, in the case of piers for

city bridges, towers for suspension bridges, or work of this character it is very desirable to have the material of pleasing appearance, and this is most readily found in the granites and marbles. The ordinary gray or bluish gray of many of the sandstones is also very pleasing, and in many cases other colors can be found for belt courses, copings, and trimmings of various kinds.

While the color of stone is apt to change considerably after it is quarried, it is usually possible to know what the change will amount to, by seeing stone of the same kind that has been in use. The gray color of many of the granites is due to a mixture of light feldspar with a dark-colored mica and very fine hornblende. The sedimentary rocks are colored with iron and various other minerals, and it is always necessary when iron is the coloring-matter to make sure that in weathering the stone will not turn rusty.

One of the most important qualities to be taken into account in selecting building stone is its durability under changes of temperature and abrasion, although the stone may prove valueless owing to chemical changes due either to its going to pieces by the action of the water, carbon dioxide, or some of the organic acids. The change of temperature affects rock by the unequal expansion and contraction of the various minerals composing it, or the rock may be so porous as to become filled with water, and when this freezes the expansion of the ice will cause it to crack or break. The report on the building stones of Wisconsin states that "the expansive force of heat is well shown in many of the limestone quarries of Wisconsin, where beds from 5 to 6 inches in thickness are for the first time exposed to the heat of the summer sun. These thin beds become heated throughout their entire thickness, and arch up on the floor of the quarry, generally breaking and completely destroying the stone." The effect of freezing and thawing is well stated in Vol. II of the Washington Geological Survey as follows: "All rocks are more or less porous; and these pores, before the rock is quarried, always contain more or less water, and after being quarried for some time and exposed to the atmosphere they lose this water. However, when rain-storms occur they are apt to absorb more water, and if the temperature falls below the freezing-point when the stone is in this condition the water will be frozen. As is well known, water on freezing expands and in expanding exerts a pressure or expansive force equal to about 150 tons to the square foot. It is plain to see that if the rock contains any very large amount of water and freezes, the result will be the spreading apart or separating somewhat of the particles composing the stone. Then the water thaws and freezes

again, and so on indefinitely, and the result is that particles are finally completely loosened and fall out. This effect is principally on the surface of the rock and is the cause of the scaling frequently seen in buildings that are built of certain kinds of stone. In addition to the pores which occur in the rocks, there are the openings which occur along the joint, bedding and foliation planes. Water falling on the surface of the ground and more or less of it sinking into it enters these cracks or crevices; and while it will flow more readily along these than it does through the pores of the rock, still many times these will be filled and freeze while in that condition. As has already been stated, water when freezing exerts a very great expansive force and will tend to separate the rocks along these planes, and when the ice melts the rocks do not come back to their original position, but retain the position they had when the water was frozen in them. Then these cracks are filled again and refrozen, and the seams opened a little farther, and the same process repeated time after time finally produces a perceptible effect and tends to weaken the stone. This is especially true of sedimentary deposits such as sandstones, particularly where they have marked bedding planes."

The effect of ice, driftwood, and the like is to abrade the surfaces of piers so that with stone that is at all soft it becomes a very serious matter. The action of sand carried by wind-storms, while very destructive, can hardly be considered in the design of piers, as it is seldom that they are so situated as to be affected in this way.

The mineralogical composition of stones has a very important bearing upon their durability, but, as this is fully treated in works on mineralogy, it will not be gone into here.

Chemical and microscopic examinations are often of value, and for any large piece of work, or where the stone as proposed for use has not been used for any great length of time, these tests should be made; but, as a general rule, the physical tests are of very much greater value. Nearly all of our universities and many of the larger engineering offices now have testing-machines (Fig. 181), so that tests can easily be made. They should be carried out by standard methods, so that they will be of value for tabulation with the results of other investigators, and of use to future consumers of the stone.

It was formerly the custom to make tests on 1-inch cubes, but wherever the testing-machine is large enough they should be not less than 2-inch cubes, having 4 square inches of area, or larger if possible, as stone in large pieces has greater resistance per square inch, and thus the actual strength of the stone will be more nearly determined.

The Wisconsin report divides the physical tests into two divisions: First, strength tests, comprising crushing strength, transverse strength—giving the modulus of rupture and the coefficient of elasticity; second, durability tests, covering specific gravity, porosity, weight of the stone per cubic foot, effect of extreme heat, effect of alternate freezing and thawing, action of carbonic-acid gas, and the action of sulphurous-acid fumes.

The crushing strength has usually been considered all that is necessary, but the above report speaks of this test as follows: " It has been computed that the stone at the base of the Washington Monument, the highest structure in the world, sustains a maximum pressure of 22.658 tons per square foot, or 314.6 pounds per square inch. Certain contractors require a stone to withstand twenty times the pressure to which it will be subjected in the wall, while others only require ten times that pressure. Even if requiring a factor of safety of twenty, the strength required for a stone at the base of this monument would be only 6292 pounds per square inch. The pressure at the base of our tallest building can scarcely exceed one-half that at the base of the monument, or 157.3 pounds per square inch. According to the above estimate, stone used in the tallest buildings does not require a compressive strength above 3146 pounds per square inch. There is scarcely a building stone of importance in the country that does not give a higher test than this. Ordinary building stone has from two to ten times the maximum required crushing strength. A stone having a crushing strength of 5000 pounds per square inch is sufficiently strong for any ordinary building." So that it will be seen that very few stones will not be strong enough in this regard.

The following tables, which are taken from the Washington Geological Survey, give a large number of tests which have been made on building stone in various parts of the country:

TABLE XXIV.—CRUSHING STRENGTH IN POUNDS PER SQUARE INCH. SPECIFIC GRAVITY, AND RATIO OF ABSORPTION OF BUILDING STONE

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
GRANITE.			
(1)*Montello, Wisconsin	43,973	2.639	.079
(1) Granite City, Wisconsin	25,000	2.675	.133
(1) Berlin, Wisconsin	32,747	2.643	.143
(1) Granite Heights, Wisconsin	16,723	2.631	.180

* See p. 260 for references.

TABLE XXIV.—*Continued.*

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
GRANITE—*Continued.*			
(2) East St. Cloud, Minnesota	28,000	2.692	2.59
(2) Sauk Rapids	21,500	2.710	.190
(2) Beaver Bay, Minnesota	20,750	2.69	.140
(3) Fourche Mountain, Arkansas	29,000	2.642	1-1673
(3) Fourche Mountain, Arkansas	28,700	2.635	
(3) Fourche Mountain, Arkansas	21,500		
(4) Little Rock, Arkansas	22,388		
(4) Little Rock, Arkansas	17,407		
(4) Millbridge, Maine	19,917		
SANDSTONE.			
(2) *Hinckley, Minnesota	19,000	2.470	4.88
(2) Dresbach, Minnesota	6,500	2.380	11.48
(2) Jordan, Minnesota	4,750	2.340	12.69
(1) Ablemans, Wisconsin	13,669		
(1) Ablemans, Wisconsin	11,030		
(1) Ablemans, Wisconsin	8,602		
(1) Ablemans, Wisconsin	10,056		
(1) Dunnville, Wisconsin	2,502	2.601	15.130
(1) Port Wing, Wisconsin	5,498	2.638	10.330
(1) Houghton, Wisconsin	4,549		
(1) La Valle, Wisconsin	13,350		
(1) Bayfield, Wisconsin	4,588	2.639	4.760
(5) Birdsboro, Pennsylvania	11,448		
(5) Waltonville, Pennsylvania	14,000	2.350	1-27
(5) Waltonville, Pennsylvania	12,730		
(5) Lumberville, Pennsylvania	22,250	2.660	
(5) Laurel Run, Pennsylvania	17,600	2.660	1-900
(5) White Haven, Pennsylvania	29,252		
(5) Portland, Connecticut	12,580	2.350	1-40
(5) Middletown, Connecticut	6,250	2.360	1-40
(5) E. Longmeadow, Massachusetts	12,330	2.480	
(5) Medina, New York	16,031	2.400	1-53
(5) Marquette, Michigan	6,150		
(6) St. Anthony, Indiana	3,000		3/40
(6) Riverside, Indiana	6,100		1/25
(6) Riverside, Indiana	6,800		1/50
(6) Worthy, Indiana	6,825		
(6) Berea, Ohio	11,213	2.110	1/20
(6) Hummelstown, Pennsylvania	12,810		
(6) Gunnison, Colorado	5,250	2.200	.09
(6) Cleveland, Ohio	6,800	2.240	1/37
(6) N. Amherst, Ohio	5,450	2.140	1/99
(6) Angel Island, California	4,574	2.730	2.730
(6) San José, California	2,400	2.640	1/16
(6) Bass Island, Wisconsin	4,850		

* See p. 260 for references.

The engineer should always make careful inquiries as to whether the manner of quarrying stone injures it in any way, as the use of

explosives will very materially shatter many kinds of stone and practically ruin them for heavy construction work.

Most all quarrying (Fig. 180) is now done by the use of drills or channeling-machines, so that damage from explosives is not so frequently found as formerly.

The larger granite quarries use no machinery in quarrying the stone other than rock drills and hoisting-apparatus. Lewis holes are drilled close together in groups of two or three, the partitions broken down, and when a series of these have been drilled around the piece to be blasted out, explosives are used to loosen the rock, and by this method the action is wedge-like, and the rock is not damaged.

TABLE XXV.

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
LIMESTONE.			
(1)*Knowles, Wisonsin	29,189	2.793	1.76
(1) Bridgeport, Wisconsin	10,112	2.740	5.49
(1) Bridgeport, Wisconsin	6,675	2.740	5.46
(1) Duck Creek, Wisconsin	23,783	2.843	.419
(1) Sturgeon Bay, Wisconsin	31,957	2.841	.19
(1) Sturgeon Bay, Wisconsin	39,983	2.700	.64
(1) Genesee, Wisconsin	36,731	2.833	1.10
(1) Genesee, Wisconsin	29,253	2.829	1.15
(1) Marblehead, Wisconsin	42,787	2.856	.31
(1) Lannon, Wisconsin	31,936	2.814	1.32
(1) Fountain City, Wisconsin	8,830	2.804	4.95
(1) Wauwatosa, Wisconsin	19,111	2.821	2.29
(1) Wauwatosa, Wisconsin	13,406	2.826	2.53
(1) Wauwatosa, Wisconsin	23,744		
(2) Red Wing, Minnesota	23,000	2.750	2.95
(2) Stillwater, Minnesota	10,750	2.590	2.19
(1) Kasota, Minnesota	18,500	2.640	2.51
(2) Mantorville, Minnesota	9,500	2.650	5.37
(2) Minneapolis, Minnesota	21,750	2.770	2.36
(7) Ellettsville, Indiana	6,700		1-31
(7) Ellettsville, Indiana	5,900		1-41
(7) Salem, Indiana	11,700	2.510	1-31
(7) Salem, Indiana	6,900		
(7) Bloomington, Indiana	4,200		1-14
(7) Bloomington, Indiana	5,700	2.460	1-19
(7) Bloomington, Indiana	8,000		1-33
(7) Romana, Indiana	7,000	2.480	1-39
(7) Bedford, Indiana	4,609	2.470	1-23
(7) Bedford, Indiana	14,000		
(7) Bedford, Indiana	6,500		1-24
(7) Salem, Indiana	8,900	2.510	1-31

* See p. 260 for references.

TABLE XXV.—*Continued.*

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
MARBLE.			
(1) Rutland, Vermont	11,892		
(1) Rutland, Vermont	13,864		
(1) Mountain, Vermont	12,833		
(1) Sutherland Falls, Vermont	16,156		
(1) DeKalb, New York	13,733		
(1) DeKalb, New York	10,478		
(1) DeKalb, New York	12,004		
(1) DeKalb, New York	13,772		
(1) Colton, California	17,783		
(1) Canaan, Connecticut	5,812		
(8) St. Joe, Arkansas	17,835	2.712	0.34
(8) St. Joe, Arkansas	10,447	2.697	0.33
(8) St. Joe, Arkansas	11,265	2.707	0.25
(8) Marble City, Arkansas	8,894	2.691	0.57
(8) Marble City, Arkansas	10,381	2.689	0.49
(8) Rhodes Mill, Arkansas	14,400	2.711	0.29
(8) St. Joe, Arkansas	6,728	2.693	0.37
(8) St. Joe, Arkansas	6,935	2.675	0.56
(8) Montgomery County, Pennsylvania	13,700		
(8) Dorset, Vermont	7,612	2.635	0.58
(8) Cararra, Italy	12,156	2.690	
(9) Georgia	10,000		
(9) Georgia	13,100	2.763	
(9) Georgia	11,400	2.717	
(9) Georgia	12,000	2.707	
(9) Georgia	10,900	2.734	
(9) Georgia	10,800		

(1) Wisconsin Geological and Natural History Survey, Bulletin No. 4, Building and Ornamental Stones, pp. 309–403, by E. R. Buckley.
(2) Geol. and Nat. Hist. Sur. of Minn., final report, Vol. I, pp. 196–200.
(3) Ann. Rep. Ark. Geol. Survey, Vol. II, 1890, pp. 44 to 50, by J. F. Williams.
(4) Tests of Metals, Government Rep., 1905, pp. 319–320.
(5) Appendix Ann. Rep., Pa. State College, 1896, p. 30 (Brownstones).
(6) Twentieth Ann. Rep., on the Geology and Natural Resources of Indiana, p. 323.
(7) Twenty-first Ann. Rep. on the Geology and Natural Resources of Indiana, pp. 313–315.
(8 (Ann. Rep. Ark. Geol. Survey, Vol. IV. 1890, p. 210, by T. C. Hopkins.
(9) Geological Survey of Georgia, Bulletin No. 1, p. 81.

In marble-quarrying channeling-machines are used, which are moved back and forth on narrow tracks and cut vertical channels 5 or 6 feet or over in depth, and a little over an inch in thickness. Some of these machines are so arranged that a channel is cut on each side at the same time. When these channels have been cut, holes are cut horizontally across the bed and the stone split loose by wedges.

Sandstone-quarrying is carried on considerably by channeling-machines, although very many quarries are still operated by blasting out large blocks and cutting them to shape afterwards, although

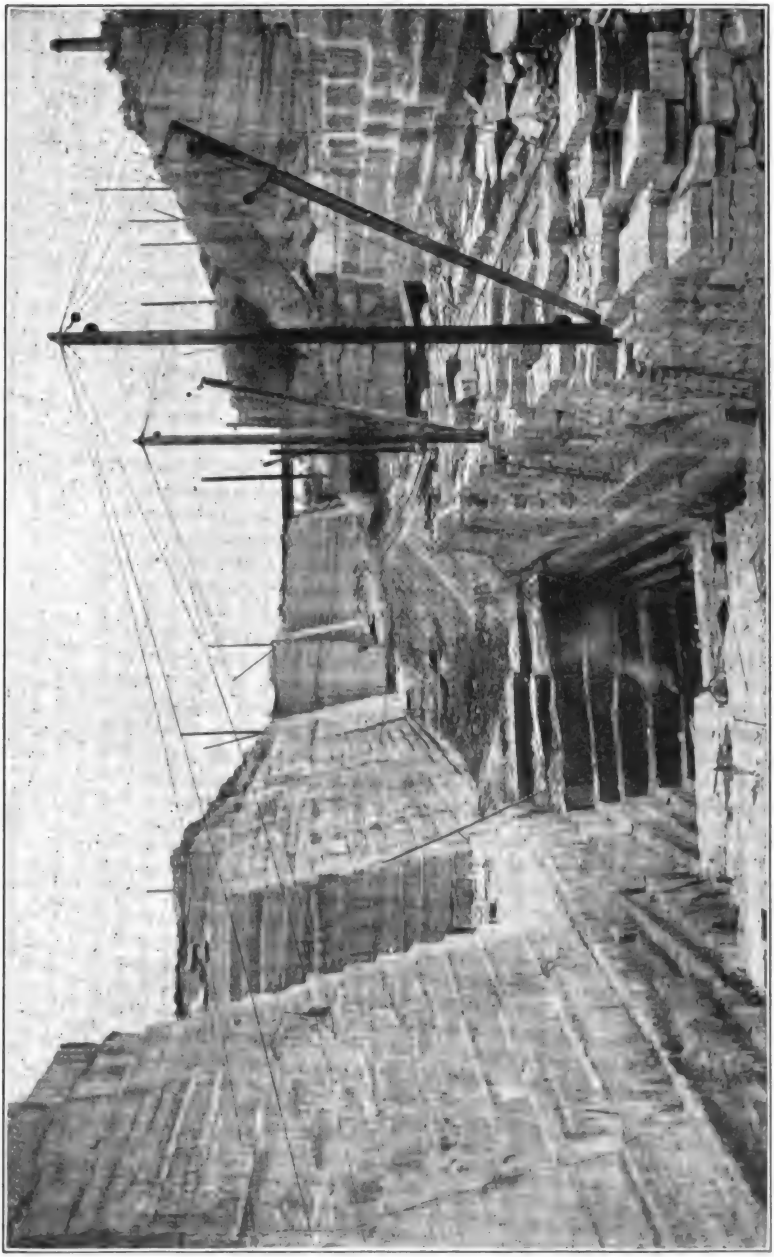


FIG. 180.—GENERAL VIEW OHIO FREESTONE QUARRY.

some damage may result to the stone from the force of the blasts. The method of working quarries is stated quite fully in the Washington Geological Survey as follows:

"The quarrying of marbles, limestones, and some sandstones at the present time is done quite largely by the use of channeling-

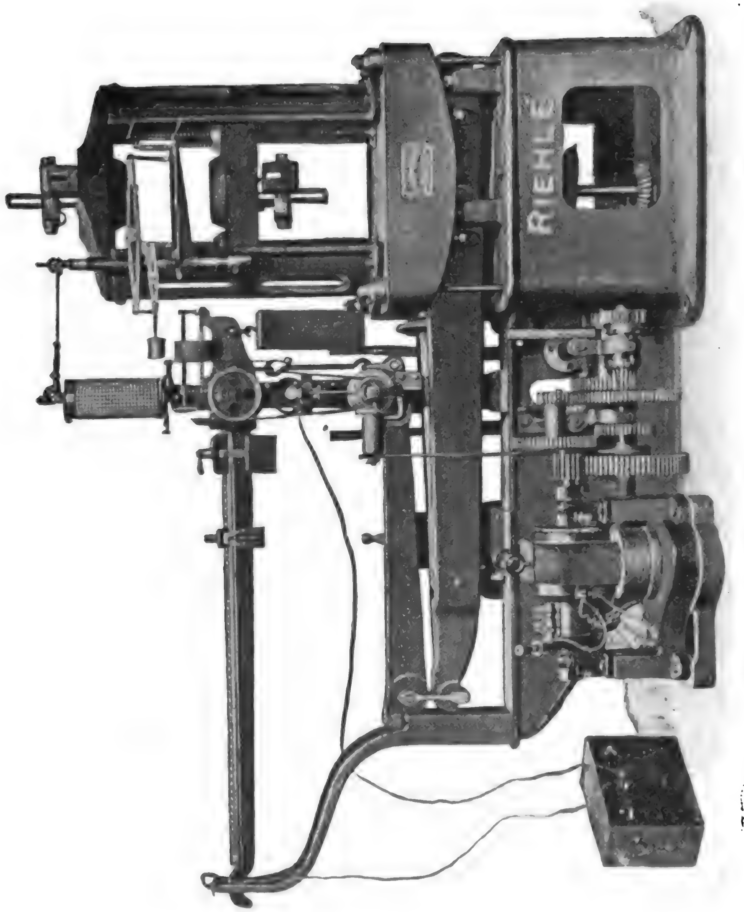


Fig. 181.—Testing-Machine.

machines of some kind (Fig. 182), while, in the harder igneous rocks such as granite, explosives are quite largely used for breaking the rock loose, after which the large masses are split and worked into sizes by hand. In the opening of a quarry in which channeling-machines are to be used, the usual thing to do is to remove the débris

overlying the stone to be quarried and secure a comparatively level floor of the same size that it is desired to make the quarry. When this is done the channeling-machine is put to work and a series of channels the required depth and distance apart are cut, and one of

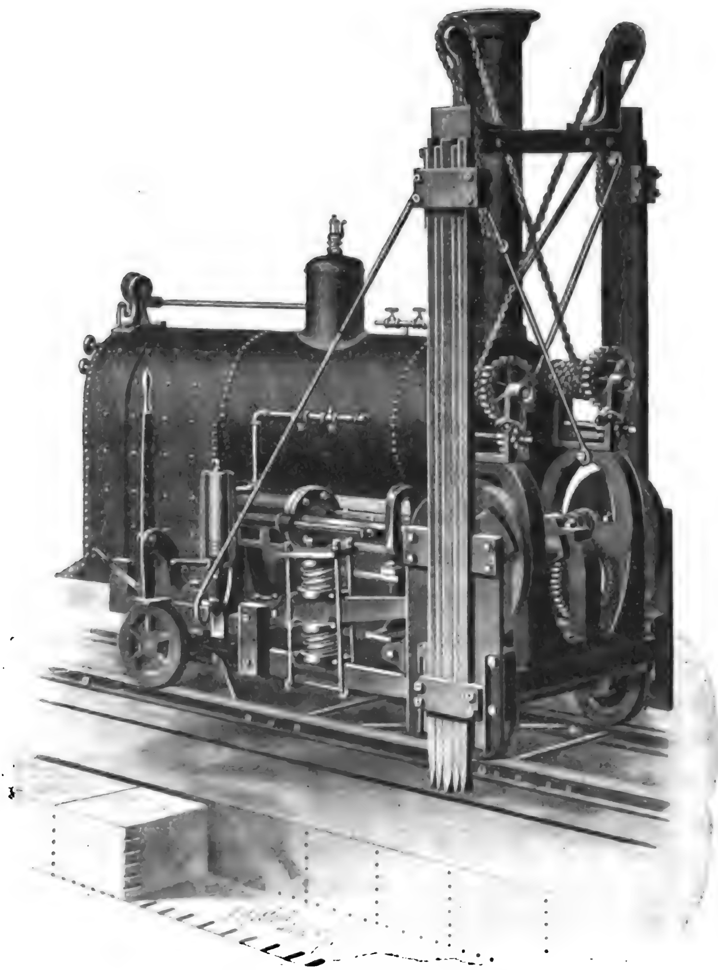


FIG. 182.—WARDWELL CHANNELER.

the blocks loosened on the under side in some manner, usually by wedging, and then lifted out, or it may be removed by blasting. After first block is removed the others may be loosened on the under side either by gadding or by means of wedges. Then another layer is begun and removed in exactly the same manner, and in this way

the quarry floor may be carried down almost any depth provided the stone continues.

"In some quarries what is known as the step or bench system is used and consists in having a ledge of varying width at the back wall each time instead of taking out an entire layer of the quarry floor. This will give to the back part of the quarry the appearance of a set of steps. If the quarry is to be worked after this plan the bar-channeler is probably the best one to purchase, as it is much more easily moved from bench to bench. In the case of quarries worked by hand, either one of the above plans may be followed.

"While many machines have been invented for cutting and dressing stone, still the same slow hand processes that were in use hundreds of years ago are still quite largely used. Large masses of the stone are loosened by means of powder and then these are split into blocks of the required sizes by what is known as the plug-and feather method. This method consists in drilling a series of holes about three-fourths of an inch in diameter and a few inches deep along a line where it is desired to split the stone. Into each one of these holes are placed two pieces of soft half-round iron called 'feathers,' and between these a steel wedge or 'plug' is placed. The quarryman then takes a hammer and moves along this line, striking alternately each one of these wedges until the stone splits and falls apart along this line. There is considerable knack in the splitting of various kinds of stone and it consists simply in being able to take advantage of the rift and grain of a stone, and it is surprising how readily some persons will work a stone into the desired shape, while others can hardly work it into any shape at all.

"In some cases stone is cut to the proper sizes in the quarry by means of channelers, steam-drills, and portable saws, but in most cases marbles, limestones, and sandstones are cut into the desired shapes after leaving the quarry and going to the mill. Usually the stone is taken out of the quarry in large blocks and then taken to the mill, where it is usually cut into the required dimensions by means of saws, and if it is to be carved or polished this is done here, and, in fact, the stone is finished ready for its place in the building.

"Most of the cutting to sizes is done by sawing (Fig. 183). This sawing is done principally by means of gang-saws which consist of a number of toothless blades of soft iron fastened in a frame in a horizontal position and this frame so arranged that it can be moved backward and forward continuously. The stone to be sawed is brought under these saws and the blades set for the required thick-

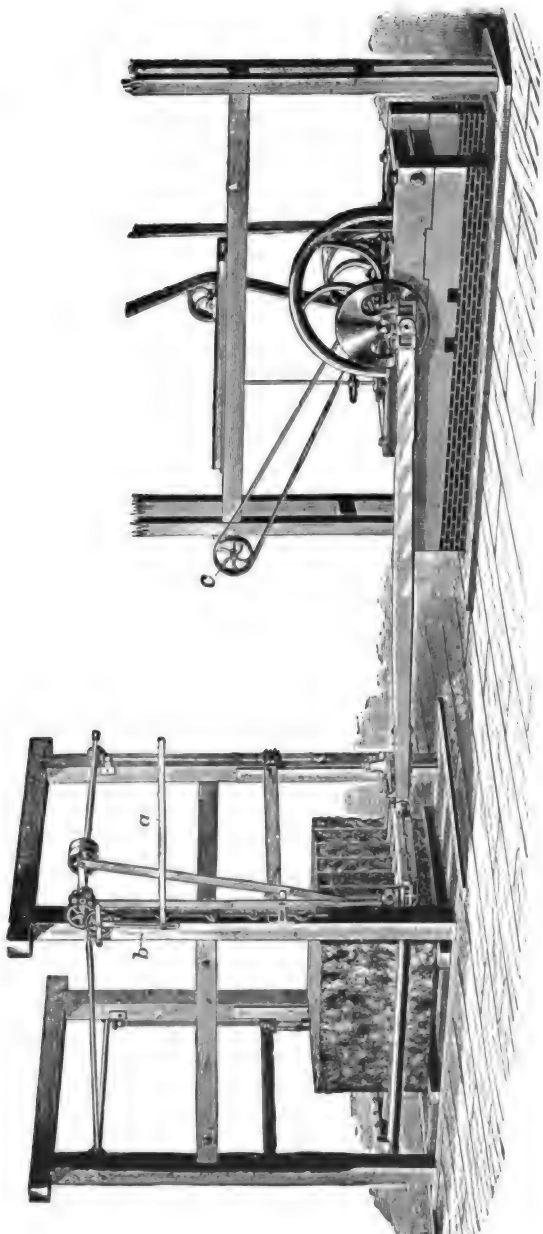


Fig. 183.—Stone Sawing-machine.

ness of the stone and then the machine set in motion. The cutting is done principally by sand or some substitute for it which, along with water, is supplied to the saw-blades. The water softens the stone, and aids in carrying the sand to the saw. The saw may be of almost any length and the frame may contain any number of blades. The blades are usually about $\frac{1}{8}$ inch in thickness and about 4 inches wide. In the latest patterns the frames are lowered automatically as the saws cut into the stone.

"The rate of cutting by these saws varies with the stone, being much faster in some kinds than in others, as, for instance, the rate for the Tenino sandstone is from 1 to 2 feet per hour, while in the serpentine, which is a much softer material, at the United States quarry the rate is not more than from 4 to 6 inches per hour.

"The kind of power used for driving these saws varies, and may be steam, electricity, or water-power, and in Washington all three are used. Steam, however, is the one most commonly used, but is much more expensive than water-power.

"Machines of various kinds for planing and dressing marbles have been constructed and they are said to work very satisfactorily, producing a surface equal to a sand-rubbed finish and saving much labor and expense in the finishing of marbles.

"Up to the present time, however, nothing of this kind is being used in this State.

"Lathes of various kinds and sizes are in use in the mills for turning marbles and serpentines. The lathes are used for turning out columns of marble and vases and ornaments of various kinds from marbles and serpentines. These lathes are practically the same as those used for the turning of wood and iron. After the desired shape has been obtained and while the column or whatever it be is still in the lathe the polisher is brought into use and it is finished before it is removed from the lathe.

"For the rubbing of the stone smooth, preparatory to polishing, the most common contrivance is perhaps what is known as the rubbing-bed. This consists of a heavy cast-iron plate which revolves in a horizontal plane. The plate is revolved by means of a perpendicular shaft through the center, which is geared to the power, this gearing in some cases being above the bed, while in others it may be below. Above the revolving bed are a number of fixed arms, extending from the center to the outside of the bed and just high enough above it so as not to touch it. The slabs and blocks of marble are placed on this rubbing-bed, and these arms prevent their revolving with it when it is put in motion. Onto this plate are then put sand

or some other abrasive, and water. A large number of pieces may be placed on one bed at the same time and rubbed at much less expense than they could be by hand."

As to the general design of piers, very little further need be said than what has been given in the preceding chapter with reference to the cross-section that produces the least resistance in a stream, and the quotation from Morison as to the form of pier adopted in the various large bridges with which he was connected.

After the loads to be carried by the pier and the weight of the pier itself have been fully determined, the size of the base of the pier can be easily determined so as to keep the pressure upon the foundation bed inside of proper limits. The unbalanced pressure, due to wind and current, must be taken account of, that from wind being calculated by the ordinary methods of moments, while the force of the current may be calculated from the discussion given in Weisbach's " Mechanics." (See Chapter XIV.)

Should a particularly large base be required, it will be necessary to offset the courses of the pier by putting in several steps, and these offsets may be calculated by the ordinary formula for transverse loading of beams. Where the pressure on the foundation bed per square foot amounts to two tons, Baker gives the following coefficients by which to multiply the thickness of the masonry course to get the offset: for granite 1.5, for limestone and sandstone 1.3, and concrete in the proportion of 1, 3, and 5, 0.3. As this is the most usual unit pressure, this will be a sufficient guide in ordinary cases. In large piers where caissons and cribs are used, they are made of the proper size to properly distribute the bearing upon the foundation bed.

CHAPTER XIV

CALCULATION OF PIERS, FOOTINGS AND RETAINING WALLS

The calculation of the stability of piers and retaining walls, and the calculation of the strength of footings is not entirely germane to the subject of this treatise, but in order to properly design and execute the foundation work it is necessary for the engineer to be able to fix the dimensions for piers, retaining walls, and footings.

Where piers have been properly designed to distribute the load on the foundation, and have been given the ordinary batter of from ½ to 1 inch per foot, there is little chance, even with the highest piers that are likely to be constructed of masonry, of any lack of stability under any ordinary combination of forces, and yet the engineer should go over the calculations for stability just as carefully as if there were real danger of failure. For wide city bridges and double-track railroad spans the investigation of stability becomes quite perfunctory, but for high piers with long spans for highway work, for lift bridges, and for single-track railway spans on high piers, the calculations should be carried out with great care in order to be positive that conditions have been fully taken care of.

The forces acting upon and lengthwise of the pier (Fig. 184) and with the current tend first, to overturn the pier; second, to slide it upon some section between the coping and the base; third, to slide it upon the base; fourth, to crush it at some section of the pier; or, fifth, to cause the failure of the foundation bed itself.

The forces acting crosswise of the pier (Fig. 185) and at right angles with the current and in line with the bridge are first, the pressure of the wind on the side of the pier; second, the force caused by the expansion and contraction of the spans; and third, the skidding force of the train. Other forces may occur, as in the case of movable or bascule spans resting on piers; collision of boats; log jams, or the lodging of heavy drift.

Taking up the forces acting lengthwise of the pier, the first effect to be considered is that of the overturning moment. The elevation of the pier shown in Fig. 184 has the points of application of the

forces shown by the letters A to G. A and B are the points of application of the wind blowing against the bridge trusses. The greatest pressure of the wind (Table XXVI) may be taken at 50 pounds per square foot on 1.8 times the exposed surface of one truss,

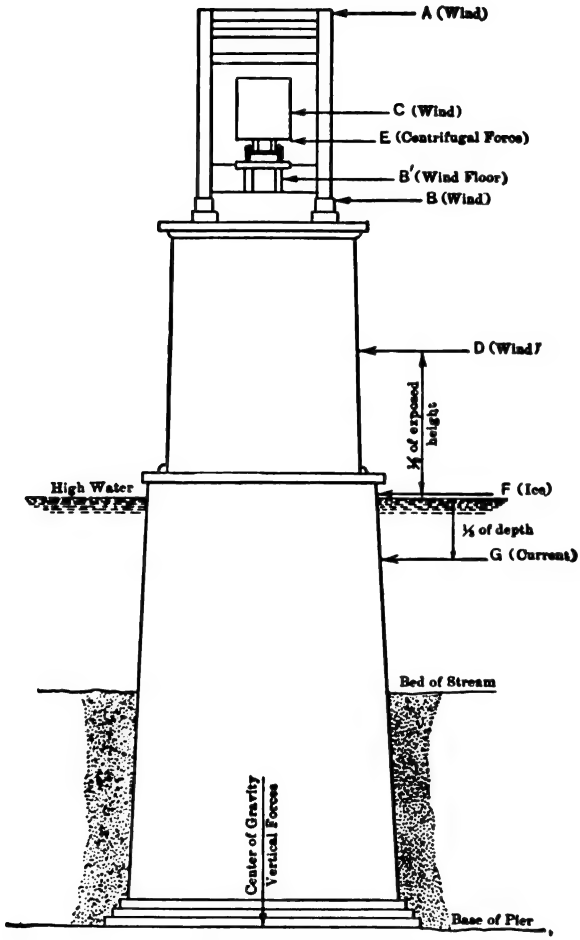


FIG. 184.—LONGITUDINAL PIER STRESSES.

or corresponding to a wind velocity of over 100 miles per hour. This pressure need not be considered as acting when a train is passing over the bridge, as it would be unlikely that a train would venture out on to a bridge in such a gale, and where the force is considered as coincident with the passage of a train it may be taken at 30 pounds

per square foot, corresponding to a wind velocity of about 80 miles per hour. These two forces comprising the total wind load on the trusses can be considered as acting half-way between A and B. In addition to these it will be necessary to figure the wind pressure on the floor system at the same pressure per square foot, acting at a point above B, at the center of the side elevation of the floor system.

TABLE XXVI.—WIND PRESSURE.

Smeaton's Formula $P = .005V^2$.

Miles per Hour.	Feet per Minute.	Feet per Second.	Force in Lbs. per Sq. Ft.	Description.
1	88	1.47	0.005	Zephyr
2	176	2.93	0.020	Very Light
3	264	4.40	0.045	
4	352	5.87	0.080	Light Breeze
5	440	7.33	0.125	
10	880	14.7	0.500	Ordinary Breeze
15	1320	22.0	1.125	
20	1760	29.3	2.000	Ordinary Gale
25	2200	36.6	3.125	
30	2640	44.0	4.500	Strong Wind
35	3080	51.3	6.125	
40	3520	58.6	8.000	Very Strong Wind
45	3960	66.0	10.125	
50	4400	73.3	12.500	Hard Storm
60	5280	88.0	18.000	Very Hard Storm
70	6160	102.7	24.500	
80	7040	117.3	32.000	Hurricane
90	7920	132.0	40.500	
100	8800	146.6	50.000	

The force C is the wind pressure on the exposed area of a train, and may be taken as acting at a height of 8 feet above the rail on a surface of 10 square feet per lineal foot of the train, or equal to 300 pounds per lineal foot of train, on the basis of 30 pounds per square foot. Complete calculations of stability should be made with and without the train.

The force D, or the wind pressure acting on the shaft of the pier, is very seldom considered, but in a strict analysis it should be taken into account and will, of course, be an amount obtained taking the width of a face of a rectangular pier, multiplied by the height above the water, multiplied by 30 or 50 pounds per square foot, as the case may be, and acting at one-half of the height from the surface of the water to the top of the pier. For a pier with rounded ends this force would only be about 0.5 of that upon the face of a rectangular pier.

The force E is the centrifugal force of the train acting at say 4 feet above the rail, where the track is on a curve. This force may be calculated from the formula:

$$F = .00001167\ V^2DW.$$

V = speed of train in miles per hour;
D = degree of curvature of track;
W = weight of the train coming upon the half spans resting upon the pier.

The force F is the pressure exerted by moving ice in rivers where ice is likely to run. This force is ordinarily figured at 300 pounds per square inch, but on the St. Louis bridge it was figured as high as 600 pounds per square inch on the area estimated to be covered by the crushing ice. Where ice or log jams occur the pressure will probably be equal to the hydrostatic pressure on the space occupied by the pier and the half span on each side, from the hydrostatic head due to the difference in the elevation of the water above and below the bridge. More than likely the force of an ice jam will be one entirely beyond the bounds of calculation, and the judgment of the engineer must govern the alllowance to be made for rivers in very cold climates; although it will probably not be necessary to go to the great extreme of building such heavy piers with ice breakers, as were constructed by Robert Stephenson for the Victoria bridge over the St. Lawrence River. The force of the ice is to be taken as acting at the level of high water.

The force G is the pressure due to the current, which may be considered as acting at one-third the depth below high water. This pressure in pounds per square foot may be taken at a value determined by Weisbach's formula:

$$P = WK\frac{v^2}{2g}.$$

W = weight of water per cubic foot = 62.43 pounds;
v = velocity in feet per second;
g = acceleration of gravity = 32.2 feet per sec. per sec.
K = 1.47 for square piers;
K = 1.33 for rectangular piers;
K = 0.73 for cylinders;
K = 0.67 for piers pointed with arcs of a circle;
K = 0.60 for elliptical piers.

The value obtained from this formula is based on the maximum velocity of the stream, and while the average velocity is very often taken at one-half the maximum, it is well to be on the safe side and use at least two-thirds of the value as obtained from the formula.

These forces acting with their respective lever arms, equal to the distance from the point of application above the base of the pier, or the section under investigation, are offset by the moment of the vertical weights, first, of the spans; second, the weight of the train (when the wind force is taken at 30 pounds per square foot); third, the weight of the pier, less the buoyancy of the water if it is possible for the water to get under the base of the pier, taken at 62.43 pounds per cubic foot, for the number of cubic feet of the pier immersed in water. Should the support afforded by the side friction of the pier be taken into account, this also should be deducted from the weight of the pier; but, on the other hand, if this is taken account of, the resistance of the ground from the bed of the stream to the base of the pier acting to prevent overturning must also be taken account of, at such a value as judgment dictates from a study of the ultimate bearing value of the soil, very small in silt and reasonably large in packed sand or gravel; so that in making an investigation of a pier's stability, a section at the bed of the stream or a small distance below, should be investigated as well as at the base of the pier.

The moment of these vertical forces is usually calculated by taking a lever arm equal to the distance from the resultant line of their action to the leeward boundary of the pier, but this lever arm should never be used, but one used of such a length to some point within the pier as judgment and careful investigation would dictate. Where the piers are founded on piles, or upon a yielding bottom, a reduced lever arm must be used, taken from the center of application of the vertical forces to the center of gravity of an area next the end of the pier, where the maximum load possible to be carried by the piles or the material would act. Where the pier is designed with a factor of safety of eight, this would reduce the lever arm by one-sixteenth the length of the pier.

The forces acting on the piers transversely, or in the direction of the center line of the bridge (Fig. 185) will consist of the forces X, Y, and Z; X being the skidding force or 0.2 times the total weight of train on the adjoining half spans.

The expansion force Y is equal to the weight of the half span expanding on a pier, multiplied by the coefficient of friction, running from 0.1 to 0.3; this will not act if large well-acting rollers are used,

but will come into play if the rollers are too small, or are rusted so they will not act.

The force Z is the wind pressure on the surface of the pier taken at 30 pounds per square foot with a train on the bridge, or 50 pounds

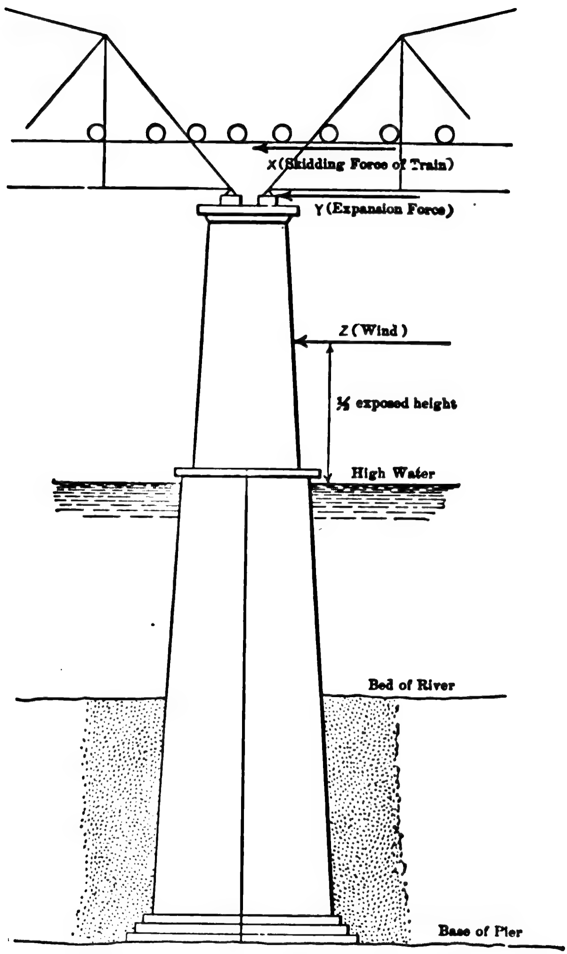


Fig. 185.—Transverse Pier Stresses.

per square foot without the train; but will not be considered to act to cause sliding if full wind pressure is figured longitudinally. The moment of these forces will be obtained with the lever arm of X and Y, from the points of application of the forces as shown, to the section being investigated, or to the maximum at the base of the pier.

The force Z is to be considered as acting at the center of the exposed area above the water; the transverse resisting moment of the pier is to be obtained in exactly a similar manner to that already described for the longitudinal forces.

The resultant of the forces in both directions tending to overturn the pier, will act to slide the pier upon any section under investigation, or upon the base of the pier, and the resistance to sliding on the base will never be less than 0.33 times the weight of the spans, train, and pier as previously determined for masonry on moist clay, up to a maximum value 0.75 of the total weight so obtained, for masonry with slightly moist mortar, at any section above the base.

The investigation into the direct crushing of the pier at any section can be carried out by determining the total load above the point to be investigated, and comparing that to the crushing value of the material of which the pier is composed. The failure of the foundation would usually only come about by the total weight on the foundation determined as above, exceeding the maximum load the foundation would carry; it should usually have a factor of safety of eight, although it should never have a less factor of safety than four against failure. When it is deemed necessary for a high pier to investigate the compression caused on the leeward side of a pier by the forces acting on it, this can be found by using the method given in Baker's "Masonry Construction," tenth edition, page 353. The pier is then considered as a vertical cantilever beam fixed at the bottom and the moment of the horizontal forces computed; then the

$$\text{Total unit pressure at the leeward side} = P = \frac{W}{S} + \frac{Ml}{2I};$$

$$\text{Total unit pressure at the windward side} = P = \frac{W}{S} - \frac{Ml}{2I};$$

W = Total vertical load on pier;
S = Area of horizontal cross-section;
M = Moment of horizontal forces;
l = Length of pier;
I = Moment of inertia of section about a gravity axis perpendicular to the horizontal forces;
I = For rectangle $= \frac{bl^3}{12}$;
b = Thickness of pier.

The allowable pressure on masonry of various kinds is given in Table XXVII, and the allowable safe load on foundations as compiled by the author is given in Table XXVIII, as supplementary to those given in Chapter XI. The calculation of the offset for footing courses has been discussed in Chapter XI, and the most valuable of the data given in the University of Illinois Bulletin No. 67 on reinforced footings is quoted verbatim at some length, but for additional information as to the calculation of reinforced footings the reader is referred to the various other bulletins mentioned in the following pages, and to the various treatises on Reinforced Concrete.

TABLE XXVII.—SAFE LOAD FOR MATERIALS IN PIERS.

Short Tons per Square Foot. Factor 8.

Material.	T. Sq. Ft. (a)	Material	T. Sq. Ft.
Granite	90	Sandstone (ord.)	50
Basalt	85	Sandstone (good)	65
Marble	75	Rubble (ord.)	3
Limestone (good)	60	Rubble (good)	5
Oolites (good)	20	Portland concrete 1 : 5	15
Brownstone (bridge)	65	Portland concrete 1 : 8	12
Brickwork (ordinary)	3	Portland concrete 1 : 10	9
Brickwork (in cement)	4	Coignet beton	15

TABLE XXVIII.—SAFE LOAD ON FOUNDATIONS.

Usual Maximum in Short Tons per Square Foot when Material is Confined.

Material.	T. Sq. Ft. (a)	Material.	T. Sq. Ft.
Fine sand	4.5	Loose beds with piling	2.0
Sand and gravel	5.0	Loose beds with concrete	3.5
Sand and clay	5.0	Brick, stock mortar	3.0
Moist sand and clay	1.5	Brick in cement	4.0
Alluvium and silt	1.5	Rock, very soft	5.0
Hard clay	5.0	Rock (hard as concrete)	10.0
Firm stone on dry clay	4.0	Rock (moderately hard)	25.0
Hardpan	8.5	Rock (very hard)	45.0

The values given for bearing power of various foundation beds in column one of Table XXVIII are the usual maximum values to be used at a depth of about 23 feet below the bed of the stream, thus making a condition of actual confinement that prevents spreading. As the foundation bed approaches the bed of the stream from this depth, the allowable reduced pressure may be obtained from the following formula by the author:

$p = 0.68a + 0.014ah$;
p = allowable pressure tons square feet;
a = constant value Table XXVIII;
h = depth below bed of stream.

Should the conditions be peculiarly good at greater depths than 23 feet below the bed of the stream, the values derived from the formula may be used up to a maximum of $1.4a$.

The subject of reinforced concrete wall footings and column footings is covered in the University of Illinois Bulletin No. 67, by Prof. A. N. Talbot. The following is taken verbatim from that paper:

Footings form an important element in the design of masonry structures. The two forms of footing most commonly used may be named the wall footing and the column footing, the former projecting laterally on the two sides of a longitudinal wall and the latter extending in four directions from the base of a column or pedestal block. It is usually assumed in the design of foundations that the earth conditions are such as to make the upward pressure on the footing uniform over its surface. Wide differences exist in methods of designing, due to differences in the assumptions made with reference to the structural action of the footing. It is not strange that these differences exist, since little or no experimental data are available which apply directly to the conditions of footings. Relatively short and deep beams and slabs under heavy uniform loads, with the supporting pressure largely concentrated at the center of the structure, may not be expected to give the same results as have been obtained in tests with the more slender beams and slabs and with the methods of support and of application of load which have generally been used in tests. With the present extensive application of reinforced concrete to footings, especially in connection with tall buildings carrying very heavy column loads, a more definite knowledge of the structural action of footings has come to be of importance. It is appreciated that the tests herein described are applicable only to a limited field, but they are offered as a contribution on a subject in which little experimentation has been done.

It may seem strange, considering the wide variations in practice, that few failures of footings have been publicly reported. It must be remembered, however, that these structures are out of sight, buried deep in the earth without opportunity for inspection. A failure in a footing may effect a change in the distribution of the load over the bed of the footing, resulting only in increased settlement. Possibly many instances of undue settlement of buildings may be due to failure in the footings. Possibly, in other cases, the earth at the center of the footing may be able to take the increased load under the conditions of side restraint developed. It is also probable that many footings have been made unduly strong.

The tests of 114 wall footings and 83 column footings are described in the bulletin. The wall footings were 12 inches wide, generally 5 feet in length and 12 inches in depth or 10 inches to the center

of the reinforcing bars, with a 12×12×12-inch stem in the middle to represent the wall through which the test load was applied. The wall footing rested on a bed of springs arranged in such a way as to approximate conditions of uniform upward pressure on the bottom surface of the footing. A variety in method of reinforcement was employed to throw light on the development of tensile stress in the steel and on the resistance to bond, diagonal tension, and shear. Tests of brick footings, unreinforced concrete footings, and footings having I-beams encased in concrete were included in the investigation of wall footings. The column footings were 5 feet square and generally 12 in. thick or 10 in. to the center of the reinforcing bars, and had a 12×12×12-in. pier built over the middle through which the load was applied. The column footings also were tested on a bed of springs which gave conditions approximating those of uniform upward pressure. Variety was given to the amount and method of reinforcement and to other conditions with a view of determining the structural action with respect to tension, bond, diagonal tension, and shear, and to give information which would bear upon methods of calculation of stresses. It is thought that these are the first experimental tests on column footings, and probably the first on wall footings on a bed of springs. Analyses are given of the stresses in wall footings and column footings and methods of calculation are discussed and compared with the results of the tests.

4. *General Theory.*—In wall footings and pier footings the weight or load is applied vertically through the wall or base block or pier, and the upward bearing pressure of the soil (which may also be called the load, since its amount and distribution determine the stresses) supports this weight from below. The usual assumption on which design of footings is based is that the soil pressure is uniform over the bed of the footing. Before uniformity of pressure on the footing will obtain, the footing must bend to the amount and form which would be caused by a uniformly distributed load. The assumption of uniform pressure is warranted if the earth layer is an elastic compressible soil of considerable thickness and of not too high a modulus of compressibility, as under these conditions the amount of bend of the projection of the footing is slight in comparison with the amount of compression of the earth. Also, in soft soils which flow laterally, as in a so-called floating foundation, the settlement and changes in the soil will produce conditions approximating uniform pressure. Where the bed is rock the pressure will be transmitted more nearly directly from the wall or pier to the rock, and as the projections of the footing have little opportunity

for being bent upward this portion of the footing may be expected to take only a small part of the load. This lack of uniformity of distribution of pressure is more likely to be present with reinforced footings than with the less flexible unreinforced footings which would carry the same load.

The principles of beam action are, in general, applicable to wall footings, but not so fully to column footings, which partake more of the nature of slabs. The formulas for calculating stresses in reinforced concrete beams have been treated in Bulletin No. 4, "Tests of Reinforced Concrete Beams: Series of 1905," and in Bulletin No. 29, "Tests of Reinforced Concrete Beams: Resistance to Web Stresses." The principal formulas for beam action in rec-

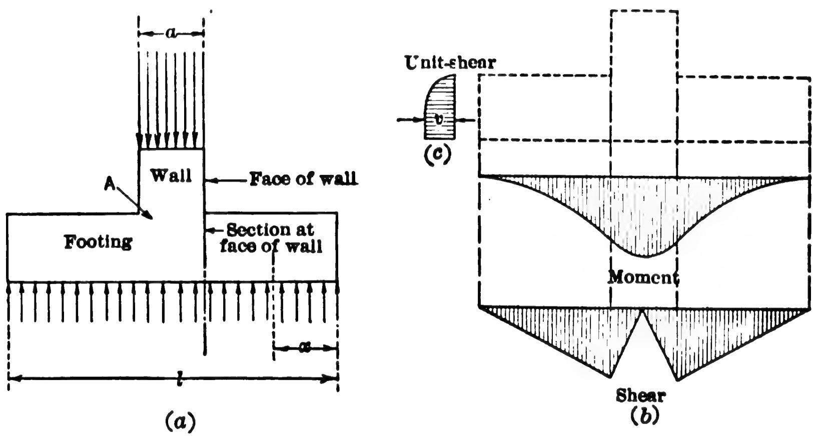


FIG. 186.—DISTRIBUTION OF LOAD AND PRESSURE IN FOOTINGS.

tangular beams reinforced for tension only, as used in this bulletin, will be repeated here.

The resisting moment of the reinforced concrete beam is (see Bulletin No. 29, page 6)

$$M = Afd' = Afjd, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

where A is the area if cross-section of longitudinal reinforcement, d is the distance from the compression face to the center of the longitudinal reinforcement, d' is the distance from the center of the reinforcement to the center of gravity of compressive stresses, j is the ratio of d' to d (which, for the beams of this bulletin, may be considered to vary from .82 to .92), and f is the tensile stress per unit of area in the metal reinforcement.

The formula for the maximum vertical shearing unit-stress in the concrete in any vertical section is

$$v=\frac{V}{jbd}=\frac{V}{bd''}, \quad \ldots \ldots \ldots \quad (18)$$

where V is the total vertical shear at the given section (equivalent to the resultant of vertical forces on one side of the section considered), and b is the breadth of the beam. This formula neglects any horizontal tensile stresses in the concrete.

The formula for bond unit-stress in horizontal reinforcing bars is

$$u=\frac{V}{mod''}, \quad \ldots \ldots \ldots \quad (17)$$

where o is the periphery of one longitudinal reinforcing bar, m is the number of bars, and the other symbols are as used before. This formula neglects any horizontal tensile stresses in the concrete.

These formulas were derived for certain assumed conditions in the beam. Since it is convenient to use them as a means of comparison for conditions other than those assumed, as, for example, when the bars are bent up at the end, the values obtained from these formulas will sometimes be referred to as nominal vertical shearing stresses and nominal bond stresses.

The value of the maximum diagonal tensile unit-stress in any section when tensile stresses exist is

$$t=\frac{1}{2}s+\sqrt{\frac{1}{4}s^2+v^2}, \quad \ldots \ldots \quad (19)$$

where s is the horizontal tensile unit-stress existing in the concrete and v is the horizontal or vertical shearing unit-stress. The direction and amount of this maximum diagonal tensile stress will vary with the relative values of s and v. In general, it may be said that in the ordinary reinforced concrete beam the value of t probably varies from one to two times v. This applies to the parts where tensile stresses exist in the concrete. Where the tensile strength of the concrete has been exceeded, it is customary to use the same formula.

It is evident that the value of the diagonal tension is generally indeterminate. No working formulas are available. For this reason it is the practice, now becoming nearly universal, in beams

without web reinforcement to calculate the value of the vertical shearing unit-stress v, and to use this as the measure or means of comparison of the diagonal tensile stress developed in the beam; with the understanding, of course, that the actual diagonal tension is considerably greater than the vertical shearing stress. It has been found that the value of v developed in beams will vary with the amount of reinforcement, with the relative length of the beam, and with other factors which affect the stiffness of the beam.

The following summary of the experiments gives the conclusions.

Wall Footings.—The tests of wall footings cover a variety of reinforcement. The method used to secure a distributed upward pressure introduced difficulties in testing. It also made it difficult to determine the load which should be taken as the critical load, and the loads which have been so specified may not always be the true critical load. The use of the bed of springs on the whole proved very satisfactory and is probably the best available arrangement for tests of the number and range used. The tests bring out phenomena which might not be apparent from analytical considerations alone or which might not be accepted without physical verification. Variations in concrete add to the complications encountered in analyzing such a series of tests. The tables and diagrams and discussions present information and data of the tests in a detailed way. The following statements summarize in a general way some of the points which are brought out by the tests and which have a bearing upon the principles and methods of design:

1. Wall footings under load follow the general laws of flexure. The section for maximum moment, the critical section for calculation of vertical shearing stress for use in judging of resistance to diagonal tension, and the method of calculating bond stress received experimental consideration.

2. The values of the modulus of rupture found in the unreinforced concrete footings are not far from the values of modulus of rupture obtained in simple beam tests such as the control beams. Increasing the richness of the mixture gives the added strength which tests of simple beams would lead us to expect. Variations in the tensile strength of concrete are to be expected, and considerable variation was found in the moduli of rupture of the test pieces, the variation being augmented by differences incident to the method of testing. The tests on footings of different lengths, undertaken to determine whether the section at the face of the wall should be used for the critical section, do not disclose any marked differences in modulus of rupture.

3. The results of the tests and the measurements of deformation of the reinforcement indicate that the critical section may be considered to be at the face of the wall and that the calculated tensile stress in the bars at this section is probably somewhat above the maximum tensile stress developed. Whether the maximum compressive stress may properly be calculated in the same way was not determined. It may be expected that high compressive stresses exist at the intersection of wall and projection. Indications of high compression and of incipient compression failure were found at the intersection of the wall and footing at loads above the critical loads.

Test pieces in which the wall was poured after the footing had taken its set, gave results which indicate that a section at the face of wall may properly be used in calculations of moments even when the wall is to be poured separately from the footing.

4. The calculations for bond stress, based upon the total external vertical shear at the section at the face of the wall and calculated by Eq. (17), evidently give stresses higher than the existing stresses. This is shown by the fact that the values calculated in this way are higher than those found in pull-out tests and beam tests. A study of the analytical conditions existing at this section tends to confirm the statement. However, as bond resistance is so important a strength element in a short cantilever beam, this method of calculation and the use of the working value of bond stress ordinarily assumed in design seems only reasonably conservative and may be recommended for general practice. Attention may properly be called to the importance of making calculations of bond stress in wall footings and other beams in which the length is short relatively to the depth. The advantage of using relatively small bars in such cases is also apparent.

Anchorage of bars by bending upward and back in a long curve or by looping the bar in a horizontal plane was found to add materially to bond resistance.

5. The tests indicate that the vertical shearing stresses developed at the face of the wall, calculated by the usual method, are higher than the vertical shearing stress which is found to exist in simple beams with concentrated loading when diagonal tension failures are developed. It was found that diagonal tension failures start at a point some distance away from the section at the face of the wall. This observation and certain analytical considerations such as the probable greater proportion of shear taken in the compression area at sections near the face of the wall show that, in calculating the vertical shearing stress which shall be used as a basis for judging the resistance to diagonal tension, a section some distance from the face of the wall

should be used. The tests and the discussion indicate that a section d distant from the face of the wall (d being the distance from the center of reinforcing bar to top of footing) may properly be used as the critical section for calculating the vertical shearing stress for this purpose, and that at this section the ordinarily accepted working stress may properly be used for calculating resistance to diagonal tension failure.

6. The bending up of bars at several points along the length of the projection gave added resistance against diagonal tension failure. Vertical stirrups also added to the resistance against diagonal tension failure but were not especially effective. Neither method of web reinforcement would be very convenient in construction. Generally speaking, it will be best to try to design the footing so that the vertical shearing stresses will be within the limit of the working stress permitted in beams without web reinforcement, and thus avoid the use of web reinforcement. In large important footings, when diagonal tension is a critical element, it would seem that some kind of unit frame with well-formed web reinforcement would be preferable to placing stirrups or to bending up bars at the necessary intervals. In stepped and sloping footings attention should be called to the larger diagonal tension and bond stresses developed. The increase in these stresses over those found in footings of uniform depth may be sufficient to decide against the use of stepped and sloping footings.

7. The footings having I-beams embedded in the concrete carried high loads, perhaps corresponding to the yield-point tensile strength of the lower flange of the I-beams and more than double what would be carried by naked I-beams. The weight of the I-beams, of course, was greater than that of the reinforcing bars used in the reinforced concrete wall footing.

Column Footings.—The requirement of uniform load and the presence of double-curved flexure complicate an investigation of column footings. In this investigation methods of testing were developed. As these are presumably the first tests on column footings, the phenomena of the tests and data of their action will be of interest to designers, especially in the directions in which tests have brought out weaknesses not always recognized and usually not guarded against. The results contribute data toward the settlement of methods of calculating of both the bending moment and the resisting moment for square footings, and the principles may with care be extended to other forms. The results may not easily be summarized, but the following statements are intended to cover the principal matters brought out in the tests:

1. A square column footing under load may be expected to take a bowl-shaped form. In slabs subject to bending in two directions, the stress in a fiber cannot differ from that in an adjoining fiber at the same level without setting up longitudinal shear; and as there is considerable resistance to variation from equality of stress in adjoining fibers, it may be expected that if stiff thick pieces (as are footings of ordinary design, where the thickness is large in comparison with the length of the projection) the deformations and consequent stresses will be distributed over the width of a cross-section and that considerable stress will be developed even in the fibers at the edge of the footing.

2. For footings having projections of ordinary dimensions, the critical section for the bending moment for one direction (which in two-way reinforced concrete footings is to be resisted by one set of bars) may be taken to be at a vertical section passing through the face of the pier. In calculating this moment, all the upward load on the rectangle lying between a face of the pier and the edge of the footing is considered to act at a center of pressure located at a point half-way out from the pier, and half of the upward load on the two corner squares is considered to act at a center of pressure located at a point six-tenths of the width of the projection from the given section. By equating this bending moment and the resisting moment which is available at the given section, the maximum tensile stress in the concrete or in the reinforcing bars may be calculated.

3. As is usually the case when plain concrete is used in flexure, the unreinforced footings show considerable variation in results. The variations were such as not to permit a method of determining the effective width of resisting section to be established or to obtain a formula for resisting moment. Based upon the full section of the footing, the moduli of rupture obtained were considerably less than the moduli of rupture of control beams made with the same concrete.

4. In reinforced concrete column footings, resistance to non-uniformity of stress in adjoining bars will be given by bond and by longitudinal shear in the concrete, and the amount of variation from uniformity of stress in the various bars will depend upon the spacing of the bars as well as upon the relative dimensions of the footing. With two-way reinforcement evenly spaced over the footing, it seems that the tensile stress is approximately the same in bars lying within a space somewhat greater than the width of the pier and that there is also considerable stress in the bars which lie near the edges of the footing. For intermediate bars stresses intermediate in amount will be developed. For footings having two-way reinforcement

spaced uniformly over the footing, the method proposed for determining the maximum tensile stress in the reinforcing bars, is to use in the calculation of resisting moment at a section at the face of the pier the area of all the bars which lie within a width of footing equal to the width of pier plus twice the thickness of footing, plus half the remaining distance on each side to the edge of the footing. This method gives results in keeping with the results of tests. When the spacing through the middle of the width of the footing is closer, or even when the bars are concentrated in the middle portion, the same method may be applied without serious error. Enough reinforcement should be placed in the outer portion to prevent the concentration of tension cracks in the concrete and to provide for other distribution stress.

5. The method proposed for calculating maximum bond stress in column footings having two-way reinforcement evenly spaced, or spaced as noted in the preceding paragraph, is to use the ordinary bond stress formula, and to consider the circumference of all the bars which were used in the calculation of tensile stress, and to take for the external shear that amount of upward pressure or load which was used in the calculation of the bending moment at the given section.

An important conclusion of the tests is that bond resistance is one of the most important features of strength of column footings, and probably much more important than has been appreciated by the average designer. The calculations of bond stress in footings of ordinary dimensions where large reinforcing bars are used show that the bond stress may be the governing element of strength. The tests show that in multiple-way reinforcement a special phenomenon affects the problem and that lower bond resistance may be found in footings than in beams. Longitudinal cracks form under and along the reinforcing bar due to the stretch in the reinforcing bars which extend in another direction, and these cracks act to reduce the bond resistance. The development of these cracks along the reinforcing bars must be expected in service under high tensile stresses, and low working bond stresses should be selected. An advantage will be found in placing under the bars a thickness of concrete of 2 inches, or better 3 inches, for footings of the size ordinarily used in buildings.

Difficulty may be found in providing the necessary bond resistance, and this points to an advantage in the use of bars of small size, even if they must be closely spaced. Generally speaking, bars of $\frac{3}{4}$-inch size or smaller will be found to serve the purpose of footings of usual

dimensions. The use of large bars, because of ease in placing, leads to the construction of footings which are insecure in bond resistance. In the tests the column footings which were reinforced with deformed bars developed high bond resistance. Curving the bar upward and backward at the end increased the bond resistance, but this form is awkward in construction. Reinforcement formed by bending long bars in a series of horizontal loops covering the whole footing gave a footing with high bond resistance.

6. As a means of measuring resistance to diagonal tension failure, the vertical shearing stress calculated by using the vertical sections formed upon the square which lies at a distance from the face of the pier equal to the depth of the footing was used. This calculation gives values of the shearing stress, for the footings which failed by diagonal tension, which agree fairly closely with the values which have been obtained in tests of simple beams. The formula used in this calculation is $v = \frac{V}{bjd}$, where V is the total vertical shear at this section taken to be equal to the upward pressure on the area of the footing outside of the section considered; b is the total distance around the four sides of the section, and jd is the distance from the center of reinforcing bars to the center of the compressive stresses. This stress is somewhat larger than the average vertical shear over the section which is sometimes used. The working stress now frequently specified for this purpose in the design of beams, 40 pounds per square inch, for 1 : 2 : 4 concrete, may be applied to the design of footings.

The punching shear may be calculated for the vertical sections which inclose the pier footing, although it may be expected that shear failure may not be produced exactly on this section. The value now generally accepted for punching shear, 120 pounds per square inch for 1 : 2 : 4 concrete, may be used for the working stress in this case.

7. No failures of concrete in compression were observed, and none would be expected with the low percentages of reinforcement used. The compressive stresses in the pier of the footing were in some cases very high and in a few instances the pier failed and was replaced by a cube of concrete. In frequent cases there were signs of distress near the intersection of pier and footing where there is an abrupt change in direction of surfaces and where the combined stresses are very high.

8. In stepped footings, the abrupt change in the value of the arm of the resisting moment at the point where the depth of footing changes may be expected to produce a correspondingly abrupt increase

of stress in the reinforcing bars. Where the step is large in comparison with the projection, the bond stress must become abnormally large. It is evident that the distribution of bond stress is quite different from that in a footing of uniform thickness. The sloped footing also gives a distribution of stress which is different from that in a footing of uniform thickness. However, for footings of uniform thickness the bond stress is a maximum at the section at the face of the pier; in a sloped footing the bond stress at the section at the face of the pier would be less accordingly than in a footing of uniform thickness, and a moderate slope may be found to distribute the bond stress more uniformly throughout the length of the bar. This is not of advantage if the full embedment of the bar is effective in resisting any pull due to bond.

9. The use of short bars placed with their ends staggered increases the tendency to fail by bond and cannot be considered as acceptable practice in footings of ordinary proportions. In footings in which the projection is short in comparison with the depth the objection is very great.

10. Footings having reinforcement placed in the direction of the diagonals as well as parallel to the sides (four-way reinforcement) gave good tests. The significance of the results is so obscured by the variety of manner of failure (bond, diagonal tension, and perhaps tension) and by variations in the quality of the concrete, that a comparison with two-way reinforcement on the basis of loads carried would not be of value. This type of distribution of reinforcement should be included in further tests. Measurements of deformation in the bars are needed to determine the division of stress among the four sets of bars.

Concluding Remarks.—The tests of wall footings and column footings leave uncertainty in some parts of the problem and there are gaps in other parts. The recent development of the portable extensometer or strain gage and the skill and experience which have been gained in its use in recent tests have opened opportunities for obtaining information on the stresses developed in such test pieces which were not available when the series of tests was undertaken. It is suggested that some of the remaining unsolved problems may most readily be attacked by measurement of deformations in the steel and concrete, and that further investigation may best be carried on by constructing a form of apparatus which will permit such measurements to be taken under the conditions of uniform loading.

The calculation of retaining walls has been the subject of numerous investigations, resulting in various formulæ, but for the purposes of

this treatise the method of calculation as given in the Engineer's Year Book has been quoted in full.

Let AB, Fig. 187, be the surface of the ground, and OB the natural slope at an angle ϕ.

Draw OE, making an angle θ, this angle being equal to ϕ plus the angle of friction of earth against the back, AO, of the wall $AOKL$.

Take X so that $EA \times EB = (EX)^2$.

Then AOX is the triangle of maximum pressure against the wall.

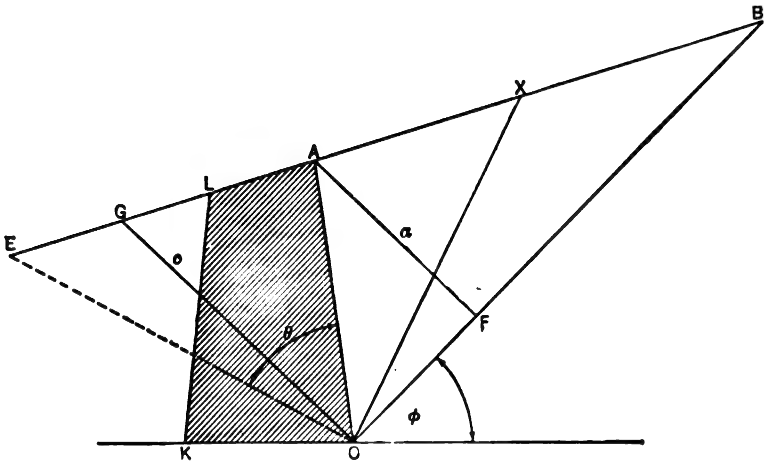


FIG. 187.—RETAINING WALL DIAGRAM.

Draw AF (a) and GO (c) perpendicular to OB, then,

$$\text{Maximum thrust} = \tfrac{1}{2}w(c - \sqrt{c(c-a)})^2,$$

where w = weight of a cubic foot of earth.

Describe a semicircle on GO (Figs. 187 and 188). Draw AD parallel to OB, cutting the semicircle at D. With G as center and GD as radius describe a circle cutting GO at P, and join DP, then,

Maximum thrust $= \dfrac{w}{2}(OP)^2$, acting at $\frac{1}{3}$ AO, and making an angle θ with the normal to AO.

Walls for Railway Cuttings

For walls over 18 feet in height: thickness at bottom $= \frac{2}{5}$ height -3 feet.

For walls less than 18 feet high: thickness at bottom $= \frac{1}{3}$ height $+3$ feet.

The two equations give the same result for a height of 18 feet. No allowance is necessary for a surcharge of less than 6 feet.

(Pennsylvania Railroad Co.)

Pressure of Water

The moment of pressure of water on a retaining wall per each foot of length of wall is $d^3 \times 10.4$, where d is the depth of water in feet. This is the moment at 1 foot. The total pressure is $31.2d^2$, assumed to be concentrated at one-third the depth from the bottom of the water. Hence the moment $10.4d^3$. The weight of the wall may be assumed concentrated at its center of gravity, and the effect

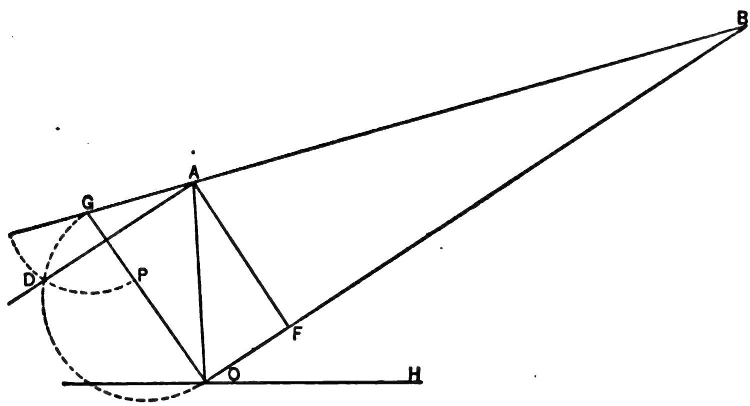


FIG. 188.—RETAINING WALL DIAGRAM.

of the pressure of the water is to turn the wall upon the point O (Fig. 189). Calling H the height of the wall in feet, and t its thickness in inches, and assuming 1 cubic foot to weigh 120 pounds, the moment of stability is $\frac{Ht^2}{2.4}$ for rectangular walls, per foot of length; or $d^3 \times 10.4$ must not exceed $\frac{Ht^2}{2.4}$; or $t^2 = \frac{25d^3}{H}$. This enables the thickness of a wall to be determined, that will exactly balance the pressure. Safety would be just secured by adding 50 per cent. to the thickness at the base, and subtracting the same amount from the top thickness, thus giving the wall a sloping back as per dotted line, increasing the acting leverage of the CG (center of gravity) of the section, so that the resultant pressure will now fall within the middle third of the wall thickness. This method is approximate and useful

for small construction. Large masonry dams require more special consideration, and may be designed by calculations at each few feet of depth, designing the wall in a series of steps and drawing a final shape to enclose the whole in one harmonious section, which must also fulfill the conditions of strength proper to the material and crushing stress. Special care is necessary that water does not get in between bed joints, as it would greatly assist to turn a wall over.

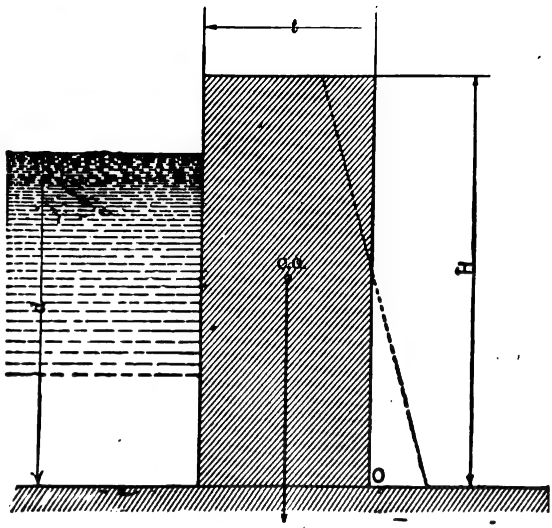


FIG. 189.—PRESSURE OF WATER ON WALL.

It is usually considered desirable that the resultant of pressure on a wall should fall within the middle third of its thickness. Thus in Fig. 190 if ab to any scale represents the total pressure $31.2d^2$ concentrated at the center of pressure upon the face of a wall, and bc to the same scale is the weight of the wall, the line bc being drawn through the CG of the cross-section, then if the diagonal of the rectangle $abcd$ cuts the base of the wall within its middle third of thickness, the wall may be assumed fully safe. If the diagonal fall outside the middle third, the thickness of the wall must be increased. It may be made safe by building the wall higher, and thus loading it; but the danger of this method is that a greater area is exposed to wind pressure, and if margins of safety are narrow the addition may be worse than useless in small work. Where a tank wall forms

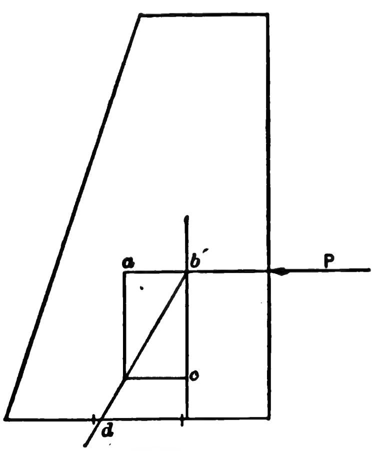


FIG. 190.—LOCATION OF RESULTANT OF PRESSURE.

a continuation of an earth retaining wall, for example, special care is necessary.

The upper part of the wall must be safe at the bed joint level with the bottom of the tank and the wall, taken as a vertical cantilever subject to a lateral thrust at a distance two-thirds of the water depth from the water surface; it must be safe against overturning at any joint lower down. It must also be safe against the added thrust of the earth behind the lower portion plus the further pressure due to the tank full of water above, which must be treated as per the rule of page 292, Fig. 193. Thus in Fig. 191 the wall must be safe on the line A against the moment of the water pressure Pc. It must be safe at the line B against the moment $(P\times d)+$ the moment $(E\times f)+$ the moment $(S\times n)$. Here E is the horizontal component of the earth pressure, and f is one-third AB; S is the horizontal component of the surcharged load, and n is one-half of AB. The safety must be proved at sufficient other depths between B and A and below B. Economy is promoted by sloping the wall against the earth between the points A and B. The wall in the figure is palpably erroneous, being far too thin for its height; it might be made safe by a sufficiency of cross-ties buried in the concrete bottom of the tank, in which case the wall from A to B must be made safe by treating it as a beam exposed to a virtual load at S and E.

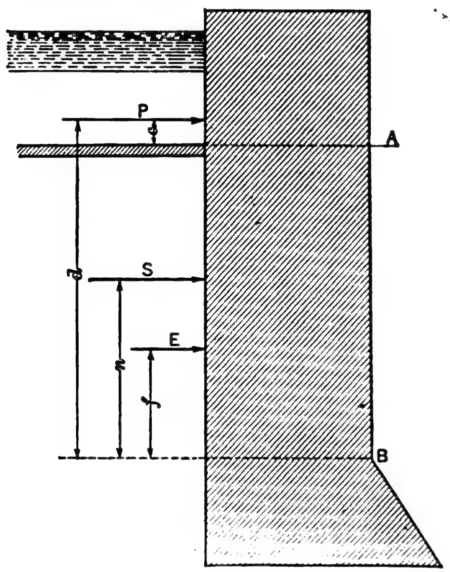


FIG. 191.—WATER PRESSURE ON RETAINING WALL.

Equilibrium of Retaining Walls

By Rankine's theory the pressure of earth on a vertical plane, AB (Fig. 192), may be considered as acting at a point, P, one-third AB from base A. and in a direction parallel to the surface. Let

w = weight of earth in pounds per cubic foot, ϕ = angle of repose of earth, P = total pressure, h = height of wall in feet.

For a bank with horizontal top

$$P=\frac{wh^2}{2}\cdot\ \frac{1-\sin\phi}{1+\sin\phi}.$$

For bank with any surface slope, θ, of indefinite length

$$P=\frac{wh^2}{2}\cos\theta\,\frac{\cos\theta-\sqrt{\cos^2\theta-\cos^2\phi}}{\cos\theta+\sqrt{\cos^2\theta-\cos^2\phi}}.$$

For bank with maximum surface slope, ϕ, of indefinite length

$$P=\frac{wh^2}{2}\cos\phi.$$

For bank with surface slope of definite length an intermediate value is taken.

To resist the pressure there is weight of wall and of earth, ABD, resting on its back, which call W; this acts vertically from center of gravity, G. The revetment may fail by overturning round f, by crushing at f, by sliding on Af.

Overturning Round f.—On any scale make $oa=W$, $ob=P$; then oc is resultant (R), acting in direction oe at e. If e falls within base, the revetment will not overturn without crushing the toe at f; but e should fall within the center third of Af.

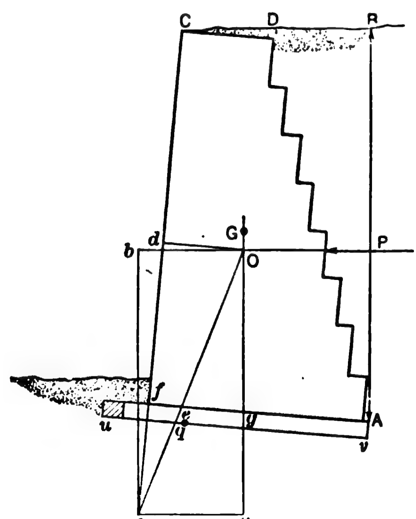


FIG. 192.—EQUILIBRIUM OF RETAINING WALL.

Crushing at f.—The resultant, R, must be resolved into two forces, cd (N), perpendicular to, and do parallel to, Af, acting at point e; the former is the crushing force.

Pressure per square inch at f on $1'$ length of wall $=\frac{2N}{fg\times 12''}$, where $fg=3fe$.

The pressure at f should not exceed power of material to resist crushing, divided by factor of safety, 4 to 8 (in practice seldom as high as 8).

Sliding on Af.—Force tending to produce sliding is od; force tending to resist it is $N\times$coefficient of friction at the joint Af; od should not exceed $N\times\frac{4}{8}$ coefficient of friction.

Surcharged Wall

If n = height of wall in feet, c = height of surcharge in feet, t = mean thickness of wall in feet, to sustain horizontal bank; t' = mean thickness of wall in feet, to sustain bank with indefinitely long natural slope, the factor of safety being about the same as for t; t'' = mean thickness in feet of surcharged wall; then

$$t''=\frac{nt+2ct'}{n+2c}.$$

Effect of Weight of Buildings on Retaining Walls

Let w_1 = weight of building per unit of surface; the rest of the notation as at commencement. The total hroizontal pressure, P, on revetment, due to the weight of building (Fig. 193)

$$=w_1(h-x)\,\frac{1-\sin\phi}{1+\sin\phi},$$

acting at $\frac{1}{2}OB$ from O; and the total horizontal pressure, P_1, on revetment, caused by the earth,

$$=\frac{w(h-x)^2}{2}.\ \frac{1-\sin\phi}{1+\sin\phi},$$

acting at $\frac{1}{3}OB$ from O.

If the front of the building were farther back from AO the effect would be less, but cannot be determined.

Foundations of Retaining Walls

Find the pressure perpendicular to *uv* (Fig. 192), the weights and pressures being taken from point *v* instead of from *A*. If *q* is center of pressure, *qu* should nearly equal *qv*, in order that the pressure may be uniformly distributed (this is unnecessary if the pressure at *u* is within what the earth can bear without yielding), and greatest pressure at *u* should not be greater than earth will bear, usually 1 to 1½ ton, or say 2500 to 3500 pounds per square foot. Foundations to be carried deep enough to avoid effects of heat and frost.

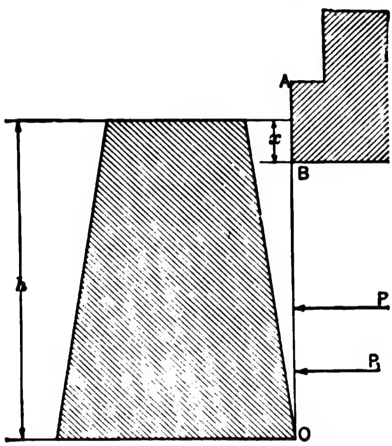


FIG. 193.—SURCHARGED RETAINING WALLS.

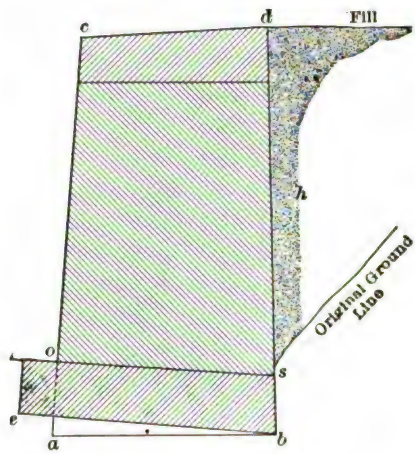


FIG. 194.—RETAINING WALL, TRAUTWINE TYPE.

The practical data as given in Trautwine, published by Wiley & Sons, are used almost universally by engineers. (See Fig. 194.)

"When the backing is deposited loosely, as usual, as when dumped from carts, cars, etc.

"Wall of cut stone or of first-class large-ranged rubble in mortar, *ab*, .35 of its entire vertical height, *db*.

"Wall of good common scabbled mortar-rubble or brick .4 of its entire vertical height, *db*.

"Wall of well-scabbled dry rubble, .5 of its entire vertical height, *db*.

"With good masonry, however, we may take the height *ds* instead of *db*, and then the above proportions of *ds* will give a sufficient thickness at the ground-line, *os*.

"When the backing is somewhat consolidated in horizontal layers, each of these thicknesses may be reduced, but no rule can be given for this.

"The offset, *oe*, in front of the wall is not included in these thicknesses."

Probably the most important points to be considered in the design of any retaining wall are the foundation and outside toe pressures, the factor of safety against sliding, the method and character of the back fill, and proper drainage.

The pressure on the foundation should be arrived at from the same data that have been given for bridge piers, and the toe of the wall is put in as shown in Fig. 195, but to be not less than what would be safe should the material become softened up on account of improper drainage. Where the ground is soft, or liable to become so, piling should be used, either driven vertically with the two rows under the toe spaced closer together (Volume II), or else with inclined brace piles under the toe as was used on a section of the seawall built at the Puget Sound Navy Yard.

In making the calculations of retaining walls the angle of repose as given in Table XXIX should be used, and the weight per cubic yard of various soils as given in Table XXX. These same values may be used to determine the force tending to slide the wall at any section or upon the base. Where the wall is not properly sub-drained, not properly back filled, and not provided with weep holes, it is probable that water pressure may be exerted on the back of the wall, and in this case the calculations must be based on the hydrostatic pressure. The author recently constructed a retaining wall from plans which called for too narrow a base, and to make it safe two rows of weep holes were provided to drain off the water, and the back filling was done with broken rock so that the water would be properly drained off. The wall also had a very considerable surcharge from the covering of large concrete pipe carried on the ground at the top of the wall, and also had to sustain the pressure from the weight of the pipe filled with water. Where a wall carries a surcharge it must be carefully calculated, but roughly speaking the wall must have a 50 per cent. wider base than an ordinary retaining wall.

The specifications of the City of Seattle, for retaining walls, published by the Civil Engineering Department of the City, are very complete, and are given in full.

Foundation.—The foundation for any retaining wall is to be excavated to the depth called for on the plan, or to such depth as the City Engineer may determine is necessary to insure a proper footing. Where the location of the wall comes on soil which, in the opinion of the City Engineer, is not firm enough to insure its safety, piling or other suitable form of sub-foundation must be placed, as

the City Engineer will direct. The foundation pits shall at all times be kept dry and free from water by pumping or otherwise as may be directed. Where permanent drainage of the foundation, or other than that shown on the plan is necessary, a suitable tile or sewer-pipe drain is to be laid and connected with the sewer or suitable outlet.

Forms.—Forms for retaining walls to be constructed in accordance with the details given on the plan, or where no details are given, in a manner satisfactory to the City Engineer. They must be constructed of sound merchantable lumber thoroughly braced and stayed, so as to produce the finished surfaces true to line and grade, and free from wind or warp or objectionable depressions and projections. Lumber used to be evenly sized and free from knotholes or other imperfections affecting the finished work. Where monolithic construction is required, particular care must be taken to construct the forms of sufficient strength to prevent bulging.

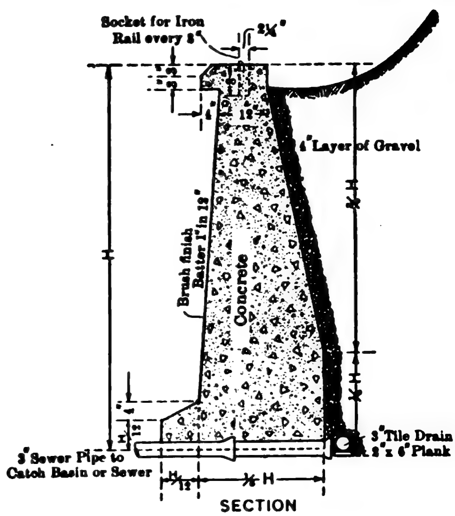


FIG. 195.—STANDARD RETAINING WALL, CITY OF SEATTLE.

All grooves, joints, mouldings, pilasters, panels and copings shall be formed true to line and dimension. Particular care to be exercised in constructing the forms for copings or other projecting parts of the wall or parapet that the same may be released and allowed to settle slightly after the concrete has partially set in order to prevent the expansion of the form from lifting or cracking the concrete at such projecting portions.

All forms to be so constructed that in stripping them from the finished work, the edges of moldings, etc., will not be defaced.

Concrete.—The concrete used in retaining walls is to be mixed in the proportions of one (1) part Portland cement, three (3) parts sand and six (6) parts gravel. The proportions of cement to the total aggregate used will be invariable, but the relative proportion of sand to gravel may be varied by the Engineer from time to time.

The concrete shall be deposited uniformly in layers but shall not be deposited in any part of the wall faster than it can be properly handled and spread into place. Depositing the material from a height into place, without properly remixing and spreading the same will not be permitted. Unless otherwise directed, the concrete must be mixed wet enough to readily spread and fill the forms but it shall not be mixed so wet that there is any tendency to wash the gravel free from the grout coating. All concrete must be thoroughly spaded as soon as deposited. The face of the wall is to be formed by spading back the gravel therefrom in such a manner as to leave a smooth cement finish. Before any concrete is deposited on top of a previous day's work, the latter shall be made rough by picking or chipping. All loose material and cement scum, or laitance, must be thoroughly removed, the surface washed clean and then grouted with neat cement. The scum, or laitance, to be removed before the concrete has set hard.

All walls shall be constructed as monoliths, where practical, that is, any section of a wall shall be deposited in one continuous operation, including the final finish at the top. Where monolithic construction is impractical, for the purpose of keeping each successive step of the work together, a recess six (6) inches deep and of a width equal to one-third the width of the wall shall be left at the end of each day's work for the entire length of such work in all walls where the cross-section is two (2) feet or more in thickness. In thinner walls, the contractor will be required to furnish and set steel dowel pins not less than three-quarters (¾) of an inch square and two (2) feet long at intervals of not less than three (3) feet for the entire length of each day's work where the same is not brought to the finished height.

In all walls the forms, moldings, etc., along the fiinished sides must be kept cleaned of any dry mortar or concrete which may mar the finished appearance.

Joints.—Joints are to be made in all walls as indicated on the plan or as directed by the City Engineer. Where joints are required the wall shall be built in alternate sections. In the ends of each completed section a recess shall be provided, four (4) inches deep and of a width equal to one-third (⅓) the thickness of the wall, but not exceeding 1 foot, for the purpose of keying the sections of the wall together, or, steel dowel pins ¾ of an inch square and two (2) feet long can be set at intervals of two (2) feet, as may be directed.

Before the intermediate sections are built the ends of the alternate

sections must be coated with one coat of expansion-joint material and four (4) layers of No. 2 tarred roofing felt, each layer of roofing felt being coated with pitch or asphalt as laid.

At the finished face of the wall, the joint shall end in a V-shaped groove two (2) inches wide and one (1) inch deep unless otherwise shown on the plan.

Finish.—As soon as the forms are stripped, the surface of the wall shall be gone over with a chipping hammer and all projections brought down to an even surface. All wires must be snipped to the surface of the wall and all holes, projections or rough spots pointed up with a mortar composed of 1 part cement to 2 parts sand. Care shall be taken in removing the forms that edges, molding, etc., are not damaged. The entire surface shall be wetted and then given a brush finish with a coat of cement grout composed of 1 part plaster of paris and 3 parts cement mixed with water to a consistency of thick cream or with a thin coat or neat cement grout, as the City Engineer directs.

Waterproofing.—The back of the wall is to be coated with tar pitch, asphalt or other approved substance. Unless otherwise directed such waterproofing will consist of two coats of the substance selected. The waterproofing must be applied hot and only on a dry surface.

Gravel.—A layer of coarse gravel not less than 4 inches in thickness shall be placed at the back of the wall for its entire height and will be paid for per cubic yard in place.

Tile Drain.—A tile drain of the size called for on the plan is to be placed at the back of the wall at the bottom and connected to the sewer where shown in the plan.

Backfilling.—The backfilling behind retaining walls is not to be made until the walls have been allowed to set two weeks or longer. The filling to be made in layers not exceeding 1 foot in thickness and thoroughly rammed. Filling in with loose earth and puddling the same will not be permitted except by express permission of the City Engineer.

Measurements.—The quantities of materials to be paid for in concrete retaining walls shall be the actual quantities in the completed work, the volumes to be determined by the prismoidal formula.

Payment for plain concrete retaining walls will include all necessary excavating, concrete, dowel pins, joints, backfilling, finishing the surface, moldings, and the furnishing, placing and removing of all necessary forms.

Piling for sub-foundation work, gravel, waterproofing, tile drain and sewer pipe will be paid for at the rates bid for the same.

In case no bid is taken for reinforcing steel, 6 cents a pound will be paid for any used.

Payment.—Payment for reinforcing steel will be in full for furnishing, bending, fitting and placing the same in the work as called for on the plan. The measurement of steel will be for the length called for on the plan or as the City Engineer may direct to be placed in the completed work.

TABLE XXIX.—ANGLE OF REPOSE OF SOILS.

(Engineer's Year Book).

Material.	Angle.	Ratio of Base of Slope to Height
Clay, dry	29°	1.8 to 1
Clay, damp, well drained	45°	1 to 1
Clay, wet	16°	3.5 to 1
Earth, dry	29°	1.8 to 1
Earth, moist	45 to 49°	1 to 1 to .87 to 1
Earth, very wet	17°	3.27 to 1
Earth, punned	66 to 74°	.45 to 1 to .28 to 1
Gravel, clean	48°	.9 to 1
Gravel, with sand	26°	2 to 1
Sand, fine dry	37 to 31°	1.3 to 1 to 1.6 to 1
Sand, wet	26°	2 to 1
Sand, very wet	32°	1.6 to 1
Shingle, loose	39°	1.2 to 1
Peat	14 to 45°	4 to 1 to 1 to 1

TABLE XXX.—WEIGHT PER CUBIC YARD OF SOILS.

Material.	Lbs.	Material.	Lbs.
Dry peat	840	Gravel	3000
Wet peat	1680	Wet sand	3140
Top soil	2240	Gravelly clay	3360
Dry sand	2470	Rough Gravel	3800
Common earth	2700	Gray chalk	4000
Sandy Loam	2700	Sandstone	4150
Marl	2910	Shale	4350
Clay	3000	Limestone	4500

CHAPTER XV

TIMBER PIERS AND TIMBER PRESERVATION

THE construction of piers of piling and sawed timber is quite common throughout the Western States, and, owing to the cheapness with which they can be built, it is often possible to construct a bridge where, if permanent foundations had to be put in, the expense would be so great as to be prohibitive. Where the piles are driven in the ground outside the water-line they will last from five to seven years, and where they are driven in fresh water they will last for a considerably longer period except that the tops of the piles will very often rot out, have to be cut off, and have blocking substituted. Cedar piles will last much longer and will not rot so quickly at the tops. Where such piers are to be constructed in salt water the piling should be protected from the teredo by some process or other, preferably by creosoting, as that will insure against both rotting and the teredo. It is also advisable in piers constructed on land, or in fresh water, to protect the piles in some way, either by creosoting or by coating them with hot carbolineum avenarius. The abutment pier shown in Figs. 196 and 197 was constructed on the line of the Puget Sound Electric Railway between Seattle and Tacoma, as the end pier to a 200-foot draw span, the bridge and foundations being constructed from plans prepared by the author. The bridge seats come directly over two piles, and these are closely flanked by two additional piles on each side, spaced 2 feet 6 inches from the center line of the trusses each way, while four additional piles are driven in the center of the pier. The piles are capped crosswise with short $12'' \times 12''$ caps, and on these are laid the longitudinal $12'' \times 12''$ caps which carry the bridge seats. The piles are braced with $4'' \times 12''$ diagonal braces boat-spiked to the piles at each intersection. The same method of construction is used for the wings, and the whole face of the pier and wings is planked up with $3'' \times 12''$ planking spiked on, with two 8-inch boat-spikes in each plank at each pile. The piling was of Washington fir with the bark peeled off and

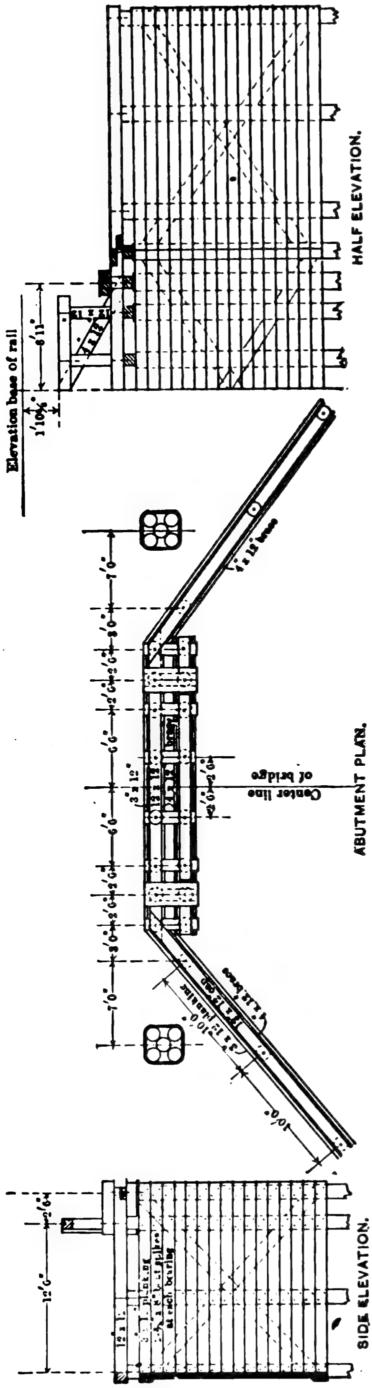


Fig. 196.—Pile Pier, Puget Sound Electric Railway.

treated with hot carbolineum avenarius. These piles were driven to a firm bearing.

The timber was No. 1 merchantable Washington fir, free from sap, wind-shake, pitch seams or other imperfections that might impair its strength and durability. In addition to the planking on the face of the pier, to protect it, five-pile dolphins were driven, as shown, 10 feet from the center of the end piles of the pier proper, and wrapped together with wire cables. Carefully constructed as these piers were, it is believed that their life will be from ten to twelve years at the least calculation. The piers, as well as the bridges, were calculated to carry train-loads consisting of Cooper's E–27 Loading. The cost of the piling was seven cents per lineal foot delivered at the bridge site, to which should be added the cost of coating, while the cost of the timber was $12 delivered at the bridge site. The cost of driving the piles was approximately $4 each for driving and cutting off, and the cost of placing the timber was approximately $4 per thousand feet board measure. Another pier on the same road is shown in Fig. 200.

In the construction of the Raging River bridge and trestle, on the line of the Northern Pacific Railroad, between Seattle and Snoqualmie

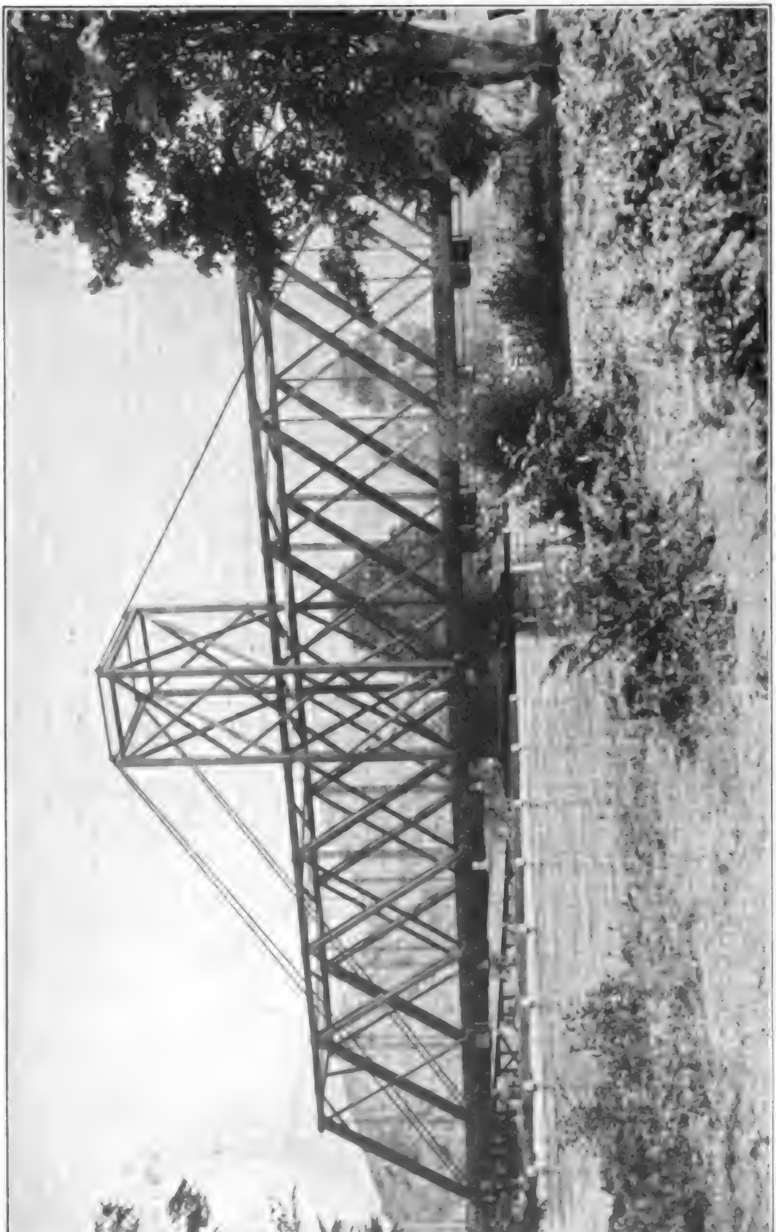


FIG. 197.—DUWAMISH DRAW, PUGET SOUND ELECTRIC RAILWAY.

Falls, it was necessary to cross the stream with a Howe truss span supported on timber towers or piers, a detail of which is shown in Fig. 198. The original trestle and bridge was left standing about thirteen years, or about three years longer than the life of the timber, and fell down during the passage of a log train. The fall was caused primarily by some logs falling off of the train and knocking out some of the batter posts of the trestle, after which the entire trestle col-

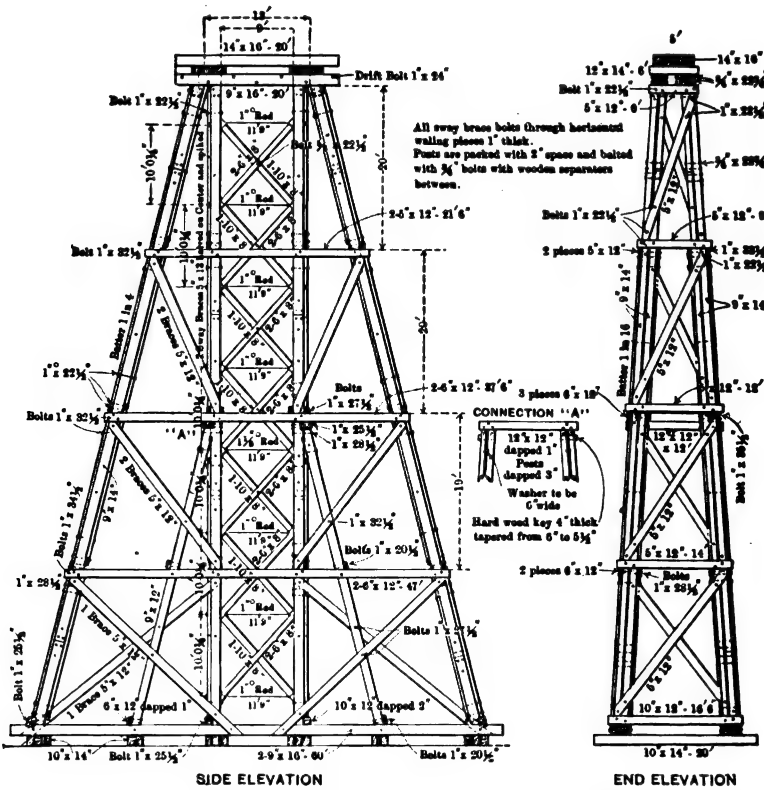


FIG. 198.—RAGING RIVER BRIDGE PIER, SNOQUALMIE FALLS LINE.

lapsed, although it had outlived its usefulness owing to the age of the timber. This occurred previous to the acquisition of the road by the Northern Pacific Railway Company. The trestle (Fig. 199) was reconstructed by the author in approximately thirty days' working time from the starting of the work until the passage of trains, although the actual time of the entire completion of the work covered a period of between six and seven weeks. All of the timber was still standing

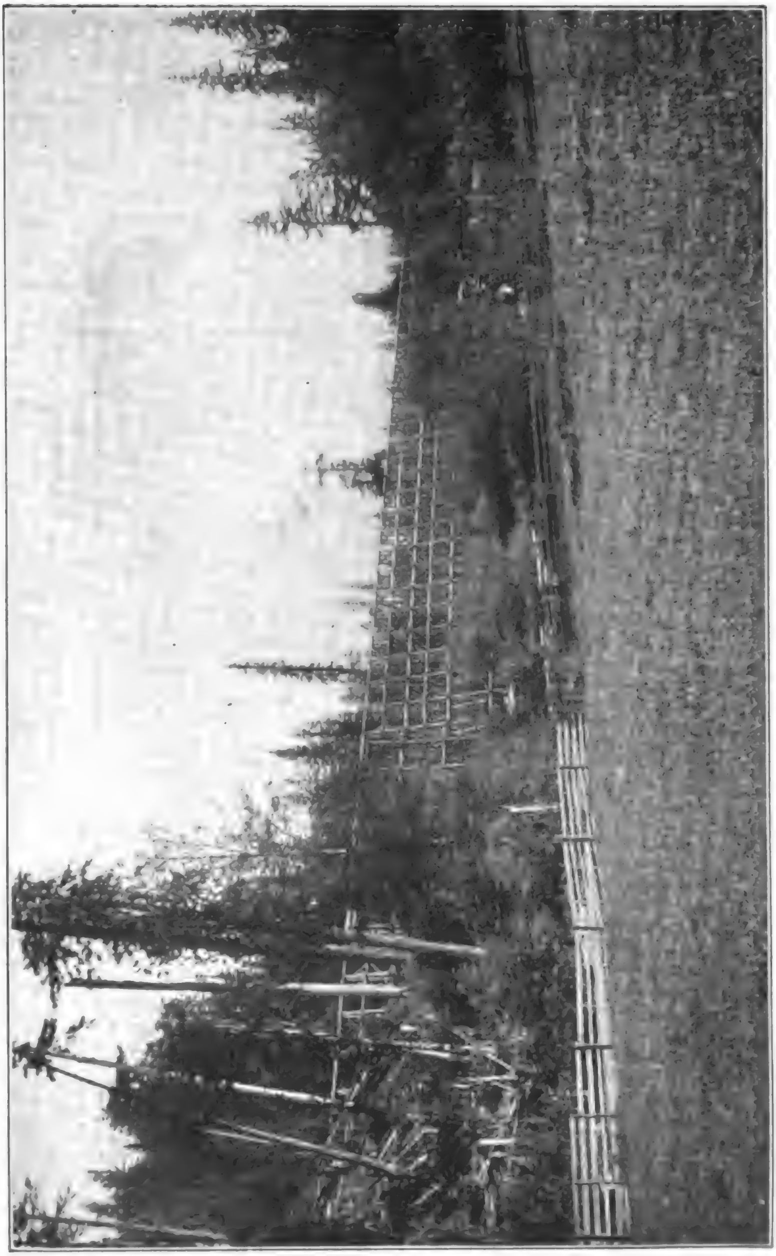


Fig. 199.—Raging River Bridge, Northern Pacific Railway.

in the forests when the contract was taken, and right of way was given the log trains carrying this timber over the road, so that it was turned out with great rapidity, and the 800,000 feet B.M. of timber that was used for the trestle and bridge was landed at the bridge site in a remarkably short space of time. The piers were carefully framed before erection was started on them, by laying them out on the ground and framing the work complete down to the boring of the bolt holes. The plumb posts were braced together transversely with Howe truss bracing, consisting of timber diagonals formed of two 6″×8″ sticks in one direction and one stick 8″×10″ in the other

FIG. 200.—PIERS OF GEORGETOWN BRIDGE.

direction. The batter posts were braced to the plumb posts with 5″×12″ and 6″×12″ girts, and 5″×12″ diagonal braces. The plumb posts and batter posts were each formed of two 9″×14″ sticks packed together. Additional batter posts 9″×12″ ran to longitudinal girts at half the height of the pier. The truss-rods of the Howe truss bracing varied from 1 inch to $1\frac{1}{8}$ inches in size, while most of the bolts used for fastening the work together were 1 inch. The tower rested upon 10″×14″ mud-sills and 9″×16″ main-sills with caps of the same size. The ends of all timbers were white-leaded, but none of it was painted or treated in any way. Such a pier as this should be good for from eight to ten years' ser-

vice for ordinary railroad traffic, although new work of this character, on even the Western railroads, at the present time is easily constructed of steel.

The designing of timber trestles, which would cover piers of the character under discussion, is most fully treated in Bulletin No. 12 of the U. S. Department of Agriculture, Timber Physics Series, the title of the Bulletin being "Economical Designing of Timber Trestle Bridges." The treatise is the work of A. L. Johnson, C.E.

The strength of the bridge and trestle timbers has been fully treated in a report by a committee of the American International Association of Railway Superintendents of Bridges and Buildings on the "Strength of Bridge and Trestle Timbers:"

"The test data at hand and the summary of criticisms of leading authorities seem to indicate the general correctness of the following conclusions:

"(1) Of all structural materials used for bridges and trestles, timber is the most variable as to the properties and strength of the different pieces classed as belonging to the same species; hence it is impossible to establish close and reliable limits for each species.

"(2) The various names applied to one and the same species in different parts of the country lead to great confusion in classifying or applying results of tests.

"(3) Variations in strength are generally directly proportional to the density or weight of timber.

"(4) As a rule, a reduction of moisture is accompanied by an increase in strength; in other words, seasoned lumber is stronger than green lumber.

"(5) Structures should be, in general, designed for the strength of green or moderately seasoned lumber of average quality and not for a high grade of well-seasoned material.

"(6) Age and use do not destroy the strength of timber unless decay or season checking takes place.

"(7) Timber, unlike materials of a more homogeneous nature, as iron and steel, has no well-defined limit of elasticity. As a rule, it can be strained very near to the breaking point without serious injury, which accounts for the continuous use of many timber structures with the material strained far beyond the usually accepted safe limits. On the other hand, sudden and frequently inexplicable failures of individual sticks at very low limits are liable to occur.

"(8) Knots, even when sound and tight, are one of the most objectionable features of timber, both for beams and struts. The full-size tests of every experimenter have demonstrated not only

that beams break at knots, but that invariably timber struts will fail at a knot or owing to the proximity of a knot, by reducing the effective area of the stick and causing curly and cross-grained fibers, thus exploding the old practical view that sound and tight knots are not detrimental to timber in compression.

"(9) Excepting in top logs of a tree or very small and young timber, the heart-wood is, as a rule, not as strong as the material farther away from the heart. This becomes more generally apparent, in practice, in large sticks with considerable heartwood cut from old trees in which the heart has begun to decay or been wind-shaken. Beams cut from such material frequently season check along middle of beam and fail by longitudinal shearing.

"(10) Top logs are not as strong as butt logs, provided the latter have sound timber.

"(11) The results of compression tests are more uniform and vary less for one species of timber than any other kind of test; hence, if only one kind of test can be made, it would seem that a compressive test will furnish the most reliable comparative results.

"(12) Long timber columns generally fail by lateral deflection or 'buckling' when the length exceeds the least cross-sectional dimension of the stick by 20; in other words, when the column is longer than 20 diameters. In practice the unit stress for all columns over 15 diameters should be reduced in accordance with the various rules and formulæ established for long columns.

"(13) Uneven end bearings and eccentric loading of columns produce more serious disturbances than are usually assumed.

"(14) The tests of full-size long compound columns, composed of several sticks bolted and fastened together at intervals, show essentially the same ultimate unit resistance for the compound column as each component stick would have if considered as a column by itself.

"(15) More attention should be given in practice to the proper proportioning of bearing areas; other in words, the compressive bearing resistance of timber with and across grain, especially the latter, owing to the tendency of an excessive crushing stress across grain to indent the timber, thereby destroying the fiber and increasing the liability to speedy decay, especially when exposed to the weather and the continual working produced by moving loads.

"The aim of your committee has been to examine the conflicting test data at hand, attributing the proper degree of importance to the various results and recommendations, and then to establish a set of units that can be accepted as fair average values, as far as

known to-day, for the ordinary quality of each species of timber and corresponding to the usual conditions and sizes of timbers encountered in practice. The difficulties of executing such a task successfully cannot be overrated, owing to the meagerness and frequently the indefiniteness of the available test data, and especially the great range of physical properties in different sticks of the same general species, not only due to the locality where it is grown, but also to the condition of the timber as regards the percentage of moisture, degree of seasoning, physical characteristics, grain, texture, proportion of hard and soft fibers, presence of knots, etc., all of which affect the question of strength.

" Your committee recommends, upon the basis of the test data at hand at the present time, the average units for the ultimate breaking stresses of the principal timbers used in bridge and trestle constructions shown in the accompanying table.

" In addition to the units given in the table, attention should be called to the latest formulæ for long timber columns, mentioned more particularly in the appendix to this report, which formulæ are based upon the results of the more recent full-size timber-column tests, and hence should be considered more valuable than the older formulæ derived from a limited number of small-size tests. These new formulæ are Professor Burr's, Appendix I; Professor Ely's, Appendix J; Professor Stanwood's, Appendix K; and A. L. Johnson's, Appendix V; while C. Shaler Smith's formulæ will be better understood after examining the explanatory notes contained in Appendix L. (The formula recommended for use by the author is nearly that of Professor Burr, or $p=6000-70\frac{l}{d}$, in which $p=$ ultimate strength per square inch, $l=$length of column in inches, and $d=$the least dimension of the column in inches. This is for Washington fir or similar timber, between the limits of $20\frac{l}{d}$ and $60\frac{l}{d}$.)

" Attention should be called to the necessity of examining the resistance of a beam to longitudinal shearing along the neutral axis, as beams under transverse loading frequently fail by longitudinal shearing in the place of transverse rupture.

" In addition to the ultimate breaking unit stress the designer of a timber structure has to establish the safe allowable unit stress for the species of timber to be used. This will vary for each particular class of structures and individual conditions. The selection of the proper ' factor of safety ' is largely a question of personal judgment

and experience, and offers the best opportunity for the display of analytical and practical ability on the part of the designer. It is difficult to give specific rules. The following are some of the controlling questions to be considered:

" The class of structure, whether temporary or permanent, and the nature of the loading, whether dead or live. If live, then whether the application of the load is accompanied by severe dynamic shocks and pounding of the structure; whether the assumed loading for calculation is the absolute maximum, rarely to be applied in practice, or a possibility that may frequently take place. Prolonged heavy, steady loading, and also alternate tensile and compressive stresses in the same place, will call for lower averages. Information as to whether the assumed breaking stresses are based on full-size or small-size tests or only on interpolated values, averaged from tests of similar species of timber, is valuable in order to attribute the proper degree of importance to recommended average values. The class of timber to be used and its condition and quality. Finally, the particular kind of strain the stick is to be subjected to and its position in the structure with regard to its importance and the possible damage that might be caused by its failure.

" In order to present something definite on this subject, your committee presents the accompanying table, showing the average safe allowable working unit stresses for the principal bridge and trestle timbers, prepared to meet the average conditions existing in railroad timber structures, the units being based upon the ultimate breaking unit stresses recommended by your committee and the following factors of safety, viz.:

Tension with and across grain	10
Compression with grain	5
Compression across grain	4
Transverse rupture, extreme fiber stress	6
Transverse rupture, modulus of elasticity	2
Shearing with and across grain	4

" In conclusion, your committee desires to emphasize the importance and great value to the railroad companies of the country of the experimental work on the strength of American timbers being conducted by the Forestry Division of the United States Department of Agriculture, and to suggest that the American Association of Railway Superintendents of Bridges and Buildings indorse this view by official action and lend its aid in every way possible to encourage the vigorous continuance of this series of

Government tests, which bids fair to become the most reliable and useful work on the subject of strength of American timbers ever undertaken. With additional and reliable information on this subject far-reaching economies in the designing of timber structures can be introduced, resulting not only in a great pecuniary saving to the railroad companies, but also offering a partial check to the enormous

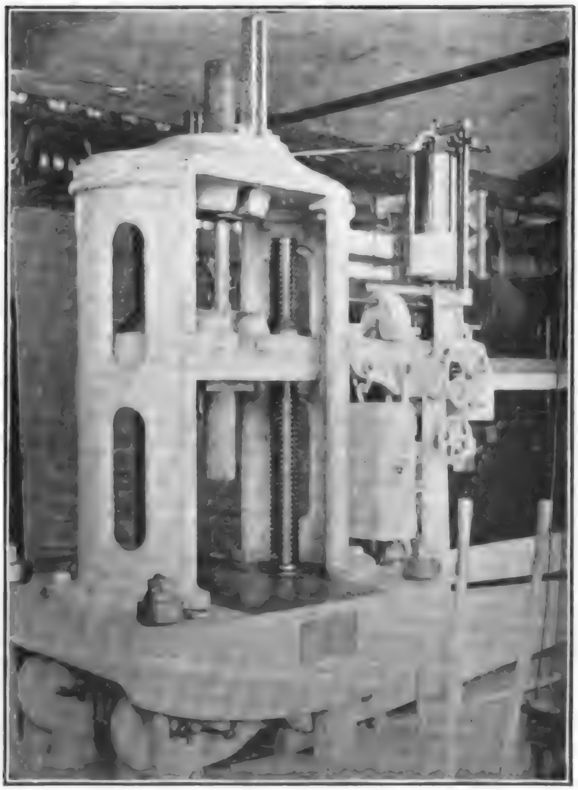


FIG. 201.—TENSILE TEST, DOUGLAS FIR.

consumption of timber and the gradual diminution of our structural timber supply."

A very complete series of tests were made by Frank W. Hibbs, Naval Constructor, U. S. N., at the Puget Sound Navy Yard, on the "Comparative Tests of Yellow Pine and Puget Sound Fir." This is fully published in a paper of the Pacific Northwest Society of Engineers, to which the reader is referred, as it is impossible to quote even results here. Fig. 201 shows the method used for making

the tensile tests, and Fig. 396 the method used for making the transverse tests. The conclusions drawn from these tests were as follows:

1st. Strength. Douglas fir is generally equal to yellow pine, and superior to it in some essential particulars.

2d. Elasticity. Douglas fir is decidedly more elastic than yellow pine.

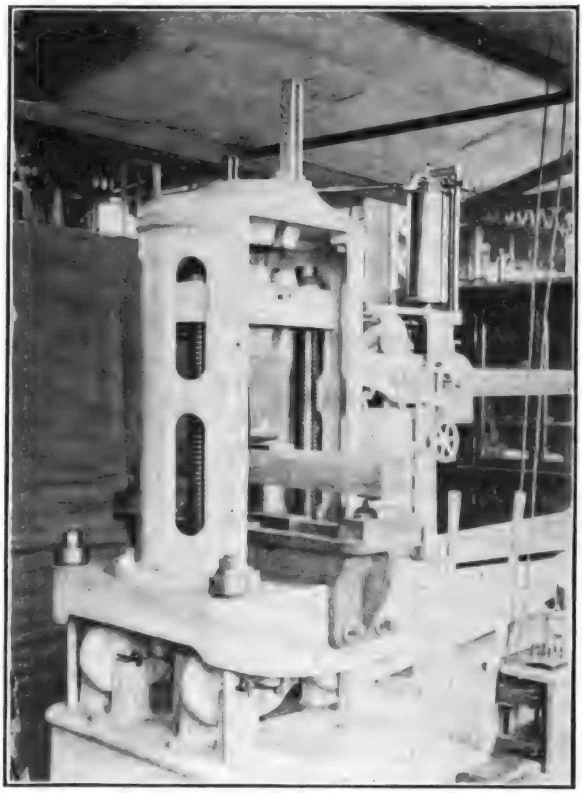


FIG. 202.—TRANSVERSE TEST, DOUGLAS FIR.

3d. Toughness. Douglas fir is far superior to yellow pine as regards toughness.

4th. Wearing qualities. Yellow pine is superior to Douglas fir, especially when moisture is present.

5th. Lasting qualities. Yellow pine is superior to Douglas fir, on account of the greater amount of pitch that it contains.

6th. Weight. Douglas fir is 14 per cent. lighter than yellow pine.

TABLE XXXI.—AVERAGE ULTIMATE BREAKING UNIT STRESSES IN POUNDS PER SQUARE INCH RECOMMENDED BY THE COMMITTEE ON "STRENGTH OF BRIDGE TRESTLE TIMBERS," AMERICAN ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS, FIFTH ANNUAL CONVENTION, NEW ORLEANS, OCTOBER, 1895.

Kind of Timber.	Tension.		Compression.			Tranverse Rupture.		Shearing.	
			With Grain.		Across Grain.				
	With Grain.	Across Grain.	End Bearing.	Columns under 15 Diameters.		Extreme Fiber Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
White oak	10,000	2,000	7,000	4,500	2,000	6,000	1,100,000	800	4,000
White pine	7,000	500	5,500	3,500	800	4,000	1,100,000	400	2,000
Southern, longleaf, or Georgia yellow pine	12,000	600	8,000	5,000	1,400	7,000	1,700,000	600	5,000
Douglas, Oregon, and Washington fir or pine:									
Yellow fir	12,000	.....	8,000	6,000	1,200	6,500	1,400,000	600	
Red fir	10,000	.....	.....	.....	.....	5,000			
Northern or shortleaf yellow pine	9,000	500	6,000	4,000	1,000	6,000	1,200,000	400	4,000
Red pine	9,000	500	6,000	4,000	800	5,000	1,200,000		
Norway pine	8,000	.....	6,000	4,000	800	4,000	1,200,000		
Canadian (Ottawa) white pine	10,000	.....	.....	5,000	.....	.....	....	350	
Canadian (Ontario) red pine	10,000	.....	.....	5,000	.....	5,000	1,400,000	400	
Spruce and Eastern fir	8,000	500	6,000	4,000	700	4,000	1,200,000	400	3,000
Hemlock	6,000	.....	.....	4,000	600	3,500	900,000	350	2,500
Cypress	6,000	.....	6,000	4,000	700	5,000	900,000		
Cedar	8,000	.....	6,000	4,000	700	5,000	700,000	.....	1,500
Chestnut	9,000	.....	.....	5,000	900	5,000	1,000,000	600	1,500
California redwood	7,000	.....	.....	4,000	800	4,500	700,000	400	
California spruce	......	.....	.....	4,000	.....	5,000	1,200,000		

TABLE XXXII.—AVERAGE SAFE ALLOWABLE WORKING UNIT STRESSES IN POUNDS PER SQUARE INCH RECOMMENDED BY THE COMMITTEE ON "STRENGTH OF BRIDGE AND TRESTLE TIMBERS," AMERICAN ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS, FIFTH ANNUAL CONVENTION, NEW ORLEANS, OCTOBER, 1895.

Kind of Timber.	Tension.		Compression.			Transverse Rupture.		Shearing.	
			With Grain.						
	With Grain.	Across Grain.	End Bearing.	Columns under 15 Diameters.	Across Grain.	Extreme Fiber Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
Factor of safety	10	10	5	5	4	6	2	4	4
White oak	1,000	200	1,400	900	500	1,000	550,000	200	1,000
White pine	700	50	1,100	700	200	700	500,000	100	500
Southern, longleaf, or Georgia yellow pine	1,200	60	1,600	1,000	350	1,200	850,000	150	1,250
Douglas, Oregon, and Washington fir or pine:									
Yellow fir	1,200	.....	1,600	1,200	300	1,100	700,000	150	
Red fir	1,000	.....	.....	.....	.....	800			
Northern or Shortleaf yellow pine	900	50	1,200	800	250	1,000	600,000	100	1,000
Red pine	900	50	1,200	800	200	800	600,000		
Norway pine	800	.....	1,200	800	200	700	600,000		
Canadian (Ottawa) white pine	1,000	.....	.....	1,000	.....	.....	....	100	
Canadian (Ontario) red pine	1,000	.....	.....	1,000	.....	800	700,000	100	
Spruce and Eastern fir	800	50	1,200	800	200	700	600,000	100	750
Hemlock	600	.....	.....	800	150	600	450,000	100	600
Cypress	600	.....	1,200	800	200	800	450,000		
Cedar	800	.....	1,200	800	200	800	350,000	.....	400
Chestnut	900	.....	.....	1,000	250	800	500,000	150	400
California redwood	700	.....	.....	800	200	750	350,000	100	
California spruce	......	.....	.....	800	.....	800	600,000		

The main reason why timber is not used in many cases is on account of its short life. Upon exposure to the elements it decays quite rapidly, lasting on the average about ten years in structures, and when used for railroad ties only about seven years. If the surface is protected from the rain by painting, this closes up the pores and causes heart rot from the moisture that is retained inside the stick of timber. For this reason paint should seldom be used upon a

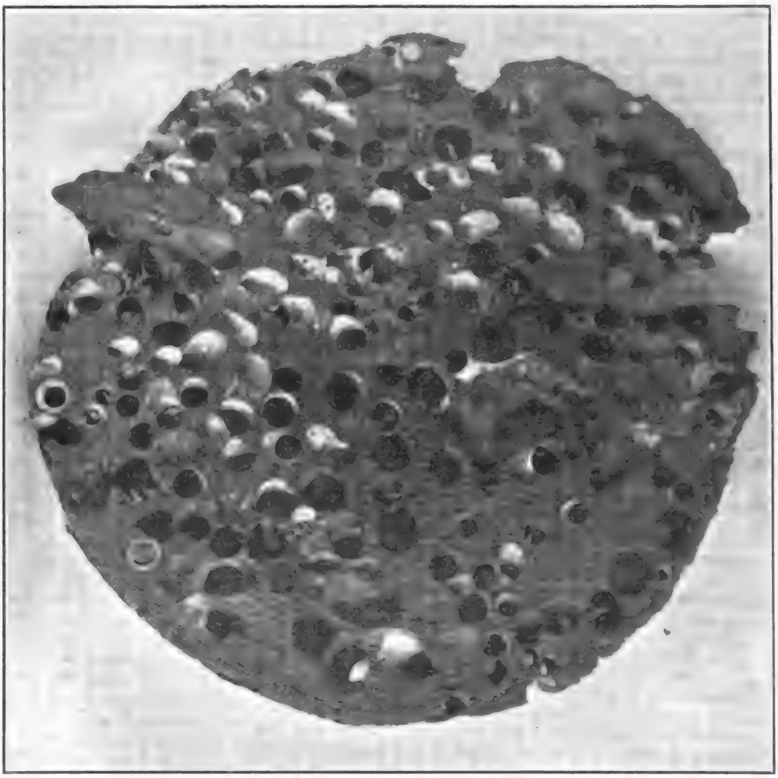


FIG. 203.—SECTION OF TEREDO-EATEN TIGHT BARK PILE.

timber structure of any kind. Where timber is placed in salt water it is destroyed from other and quite different causes. The marine animals which are most destructive are the limnora, which eats off the surface of the timber near the water line, and the teredo, which eats out the interior. Untreated peeled piling driven in salt water become so badly eaten by the teredo in two or three months as to break off under almost no load at all, if this load be applied transversely. The view, Fig. 203, is of a section of piling which was eaten

off in less than two years, it having been driven with the bark on. The bark is a good protection, and some of the methods of protecting piling against the teredo are based upon the plan of providing piles with an artificial bark formed of burlap, treated with asphaltum and wrapped about the piles with wires. The best protection, however, is creosoting, or the impregnating of piling or timber with creosote or the dead oil of coal tar.

The amount of creosote used varies from 10 to 20 pounds per cubic foot, from 12 to 16 pounds being the ordinary amount specified. For the preservation of timber simply against rot, 10 or 12 pounds is sufficient, while many engineers think it necessary to use from 16 to 20 pounds for protection against the teredo. Soft timber takes creosote readily, but some of the harder kinds, or those with hard fibers, will not take up the creosote so easily, so that it would seem advisable to specify the amount of penetration the creosote should have from the surface of the wood inwards instead of specifying the amount per cubic foot.

A penetration of five-eighths of an inch forms a satisfactory protection against the teredo.

As the other methods of preserving timber are less used, they will only be mentioned, and in case the engineer should find a plant available for preserving timber by one of the other methods its processes can readily be compared with those of creosoting. These methods are Kyanizing, or bichloride of mercury process; Burnettizing, or zinc chloride process; and Margaryizing, or sulphate of copper process. None of these, however, is so good as creosoting, inasmuch as the preserving material dissolves out of the timber and leaves it to decay. This feature is very strongly brought out in some experiments that have been made with Egyptian mummies, as when all of the embalming material has been extracted, the mummies at once decay rapidly.

Creosoting plants are found in almost every section of the country, and a description of the methods followed at any one of these works practically covers the methods employed at others.

The description given by the Norfolk Creosoting Company is quoted here in full:

"The preservation of timber by the dead oil of coal-tar process, as carried on by all well-equipped creosoting plants, consists of two distinct operations—the preparation of the wood, and its impregnation with the preservative. The preparation of the wood necessary for the proper reception of the preserving substance is the removal of all those portions of the tissue which are subject to fermentative

action. This consists of the extraction of the liquids and semi-liquids occupying the interfibrous spaces, and constituting the very immature portions of the wood, without softening the cement binding of the fibrillæ, or bundles of cellulose tissue, forming the solid or fully matured part. Upon the successful accomplishment of this entirely depends the value of artificially preserved wood for structural purposes. If this step of the operation is conducted at too low a temperature, or for too short a time, the sap or liquid part nearest the surface will only be extracted, the consequence of which will be an insufficient space for receiving the preservative. If, on the other hand, the operation is carried on at too high a temperature, or for too long a time, the resinous portion of the bundles of fibrillæ will be softened and the wood lose its elasticity in just the proportion that the coherence of the fibrillæ is lessened. The temperature should never be less than 100° C. nor exceed 130° C. Of the two possible methods for the removal of the undesirable portions of the timber—exposure to currents of dry air, and steaming under pressure with an after-drying in a vacuum—the latter is now the universal practice. While the first-named plan may seem the more rational, and the one least likely to modify injuriously the physical structure, such is not the case. Under proper manipulation, a more thorough desiccation, without harmful change of the organic structure, can be accomplished in twelve hours less by the latter process, than is ever possible with air drying, which, under the most favorable circumstances, is a long-drawn-out operation, and cannot do more than extract the water from that portion of the sap which has not yet reached the semi-solid stage, thus leaving in the tissues of the wood a very considerable amount of resinous matter which occupies space that should be ready to receive the creosote-oil. The consequence of this is a failure of the oil to reach many of the interfibrous passages, which are either left empty or are filled with the gelatinous part of the half-matured growth cells in which are to be found the conditions that make putrefaction possible. In order to remove the sap from wood, it is first necessary to vaporize it and then to bring about such external circumstances which shall allow outflow of all gaseous matter from the interior of the wood. In order to vaporize the sap it is necessary to break down the walls of the cells containing the liquid and semi-liquid substances. This is readily accomplished through the agency of heat applied through the medium of a moist steam-bath, at such a pressure as to keep the temperature of the wood, and its surrounding atmosphere, somewhat above the boiling-point of the sap. The maintenance of this condition for a few hours is found to be quite

sufficient to break down the sap-cell tissue and to vaporize all those constituents that it is desirable to withdraw. This point having been reached, the steam-bath is discontinued; and the temperature being maintained at, or slightly above, the vaporizing-point of the sap, the pressure of the atmosphere surrounding the wood within the chamber is reduced below that of the interior of the wood. The result of this condition is an outflow of vapor and air, continuing until equilibrium is restored. This equilibrium is prevented by the use of an exhaust pump until the absence of aqueous vapor in the discharge from the pump indicates the completion of the operation. At this stage the wood-tissue is in a state very like that of a sponge cleared of hot water; every pore is gaping open and ready to receive the oil.

" In the practice of the Norfolk Creosoting Company the most carefully dried lumber is steamed and subjected to the action of the heated ' vacuum ' in order that there may be had that thorough and uniform penetration of the preserving liquid that is essential to the highest efficiency of the product. The timber having been thus prepared the creosote-oil is admitted to the chamber, which is still kept under the influence of the vacuum pump, at a temperature somewhat above the boiling-point of the sap, at the pressure then existing in the chamber. As the hot oil envelopes the wood and enters the interfibrous spaces, the aqueous vapor yet remaining in the wood, by reason of its less specific gravity, rises to the top of the containing chamber and is withdrawn by the pump. By the time that the chamber is entirely filled with oil, all the remaining moisture has escaped. The exhaust pump is stopped and, in order to facilitate the absorption of the oil by the wood, a pressure pump is set to work supplying oil to the chamber at such pressure as may be desired. This operation is continued until the requisite amount of oil has been put into the timber. The chamber is then opened and the timber withdrawn. The apparatus is then ready for further use.

" The successful conduct of the operation above outlined exacts the most careful attention and skillful management, supplemented by adequate and suitable applicances. The wide divergence in the characteristics of timber; the varying amounts of sap, due to the lapse of time since, and the season in which the tree was felled; its possible subsequent immersion in water for a longer or shorter time; the character of the soil and the conditions under which the tree grew, whether in a dense forest or a comparatively open country, whether it is of a rapid, even growth, or a slow intermittent one, are all factors contibuting to a more or less perfect product. To the

experienced operator these conditions indicate, in each case, the proper course to be pursued. Failure to observe and to take them into consideration is to invite indifferent, uncertain and in the end unsatisfactory results. Of equal importance is a proper understanding

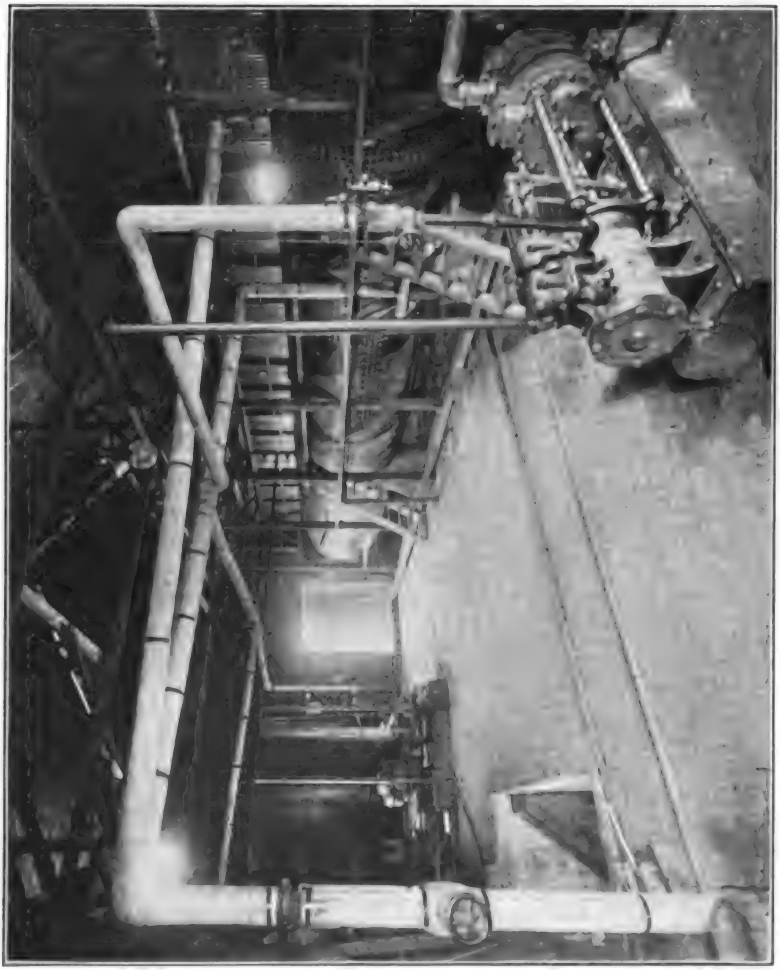


FIG. 204.—PLANT FOR CREOSOTING TIMBER.

of the circumstances under which the finished product is to be used. Timber for piers, wharves and other structures in tropical waters demand processes and degrees of thoroughness of treatment that are unnecessary in the harbors of more temperate climates, which are, in turn, more exacting than land and fresh-water construction.

"It is as true as it is unfortunate, that, in the past—perhaps

at present—much creosoted work has fallen far below the reasonable expectation of the purchaser and user. As creosoting is neither a secret or patented process, nor are its operations complex, a close and systematic inspection of materials used at the place of manufacture

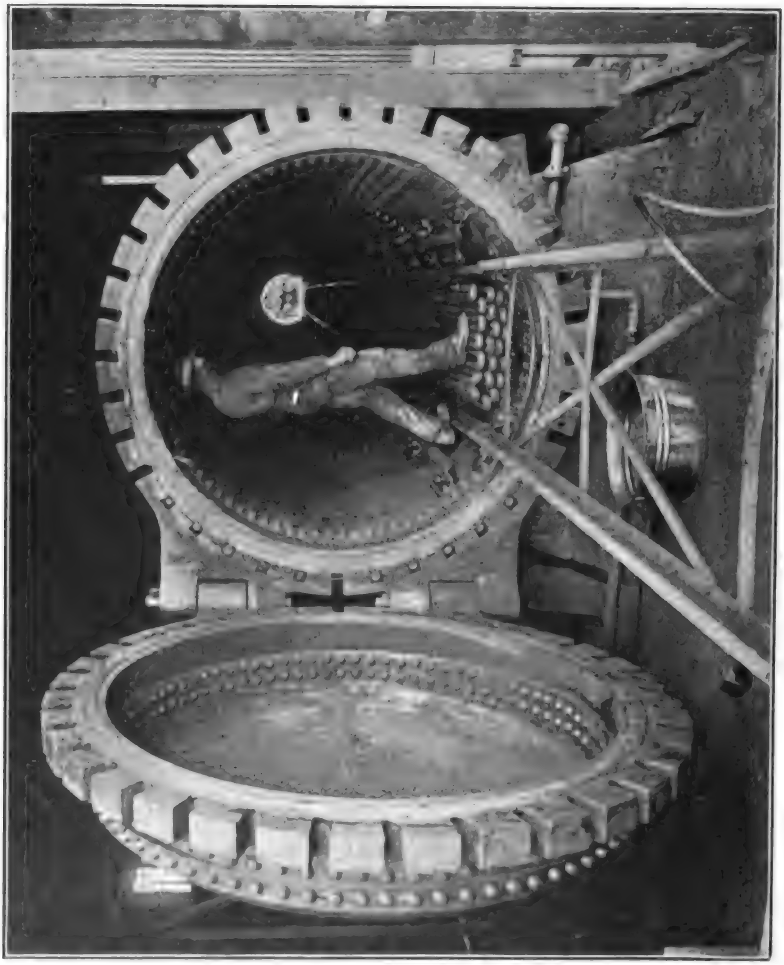


FIG. 205.—CREOSOTING RETORTS.

is all that is necessary for the buyer, and at the time that the creosoting is in progress."

The cost of creosoting varies of course with the size of piling and the size of timbers and with the location as well, but as a general average the cost of treating ordinary sized piling with 14 pounds of creosote per cubic foot will add about thirty-five cents per lineal

foot to the price of the piling, and the cost of creosoted trestle timbers will range from $35 to $55 per thousand feet B.M. in addition to the price of the timber. Views in a modern creosoting plant are shown in Figs. 204 and 205, which plant is located near Seattle, Wash. The retorts are built strong enough to make it possible to fill specifications requiring large percentages of creosote to be put in fir timber, which is much harder to treat properly than other kinds.

The specification for the boiling process of treating timber with creosote, is given in full in Appendix X, by permission of F. D. Beal, Manager of the St. Helen's creosoting plant at Portland, Ore., and is a full description of this valuable method for hard-fiber timber.

Recent experiments on Douglas fir, both treated with creosote and without treatment, indicates that the creosoted timber is considerably weakened by treatment, the strength being reduced from 15 to 30 per cent.; the length of time taken for treatment and the amount of heat to which the timber is subjected being the largest factors affecting its strength. Air seasoning after treatment does not restore the strength of any appreciable degree.

The experiments are being carried out at the University of Washington by O. P. M. Goss of the Forestry Bureau, and the full report of the tests will soon be made public in a government bulletin.

CHAPTER XVI

RETAINING WALLS AND CULVERTS

THE theory of retaining walls has been treated to some extent in Chapter XIV, where Rankine's theory of conjugate pressures has been given, and also some practical rules for proportioning retaining walls. The theory of the maximum wedge as propounded by Coulomb is also given, and can be found more fully discussed in many of the standard text-books on mechanics; so that little further will be attempted in this chapter than to discuss the practical design of retaining walls. The very thin wall as designed by French engineers for a road in Algiers is described later, as showing the limit of thinness for reinforced concrete wall construction. Solid masonry or gravity walls are often constructed by Continental engineers with bases of from only one-fourth to one-third of the height, but when such slight dimensions are employed then the foundation and back filling must be of the best, and the drainage most carefully arranged.

The careful application of theory will not always prevent failures, and the most carefully made design must be the result of a thorough study of local conditions, together with a comparison of the proportions as determined by practical rules, such as those of the Pennsylvania Railroad, which are given in this chapter. The somewhat light proportions as established by the City of Seattle and as given on page 295, will usually be found sufficient if all the precautions already given are observed. The appearance of a retaining wall can be greatly enhanced by adopting a varying batter for the face, similar to the cross-section of the Antwerp quay wall described in Chapter XXIX, Volume II.

The investigations for any given case must cover not only the safety against overturning, but must also be extended to make sure that the wall will not slide upon its base, nor crush the masonry footings, or foundations; in conformity with the principles established on pages 291 and 292.

The amount of the overturning force and its point of application having been determined from the method and equations given on page 287, the moment of this force about the outer toe of the wall

must be less than the moment of the weight of the wall acting vertically through its center of gravity about the same point, or else the wall will overturn.

The wall will be safe against sliding if the resultant of the overturning force and the weight of the wall, acting through the center of gravity of the wall, falls within a point on the base as determined by drawing a line through the center of gravity of the wall and having an angle with the vertical, equal to the angle of friction of the masonry on the foundation.

The wall will be safe against crushing when the pressure at *f* in Fig. 192 does not exceed the allowable crushing or bearing value of the material.

The general principles of design to be observed may be taken as stated in the following paragraphs.

The overturning moment depends upon the following features:

1. The weight of the fill or embankment retained.
2. The angle of repose or internal friction of fill.
3. The height or amount of surcharge.
4. The height of the retaining wall.
5. The shape of the retaining wall.

The resisting moment depends upon the following features:

1. The weight of the masonry.
2. The width or base of the foundation.
3. The shape or cross-section of wall.

The most economical section of a retaining wall would be when the back slopes towards the filling, but such a wall would usually heave from frost, so the batter in cold climates should always be reversed.

The batter of the front face of a wall should be 1 in 12, 1 in 24, or else a varying batter as used in the Antwerp sea wall referred to above. The toe of the wall or footing course should be extended to a distance, so that the center of gravity would be back of the center of the base or footing course.

The cross-section of a reinforced concrete wall, when the right of way prevents the extension of the toe, should be L shaped, and then the weight of the filling will increase the stability. But when the bank is hard to excavate, the L of the wall should be reversed if the right of way permits of the necessary room.

Light French Reinforced Wall

The improvement of the national highway from Algiers to Mostaganem at the crossing of the Ferrah ravine, was carried out to remove the dangerous curves and steep grades up to 6.5 per cent. The work required a fill of 6.5 meters above the previous filling. A retaining wall was necessary to retain the fill, which had to be constructed for a greater part of its length on the old fill. It also was necessary to provide a subway under a railway siding. The foundations of this under passage were placed on the same filled ground. The retaining wall and the underway were both built of concrete from the plans of Considere, Pelnard and Caquot.

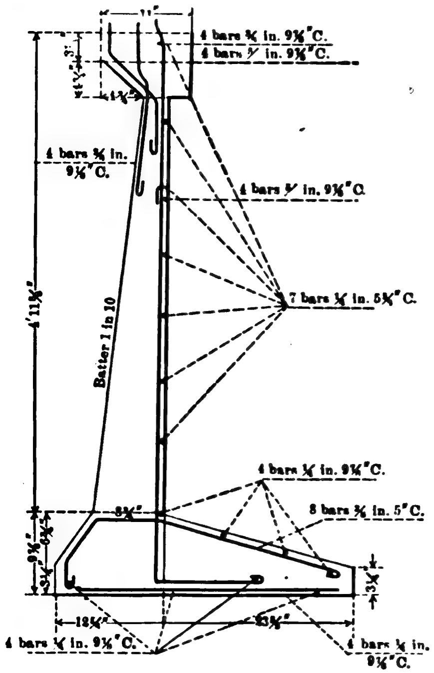


FIG. 206*a*.—RETAINING WALL, ALGIERS.

The retaining walls of reinforced concrete have a total length of 151 meters; the first 25 meters near Algiers and the last 7 meters near Mostaganem are founded on rock; but the remainder is on the old filled-in ground. Where the height does not exceed two meters, the wall consists of a vertical part with a thickness at the top of 0.08 meter, with a batter on the face of one-tenth, and being of the cantilevered type with a base of reinforced concrete as shown in Fig. 206(*a*). When the height of the wall is over 2 meters, the vertical part has a thickness at the top of 0.08 meter, with a batter of one-tenth, and has a base of from 0.10 to 0.20 meter thick and from 0.30 to 1.20 meters wide, these dimensions being proportioned to the height of the wall and the nature of the ground (Fig. 206(*b*)).

To resist the overturning moment, the wall is ribbed and braced by buttresses on the inside, consisting of one brace from the bottom and another from a point one-third the height of the wall from the top, these braces being 2.50 meters apart. Where the wall is on filled-in ground, it is divided into panels 5 meters long, which can act independently to prevent the wall from cracking. Each panel

has two main buttresses 2.50 meters apart and ribs 1.25 meters apart.

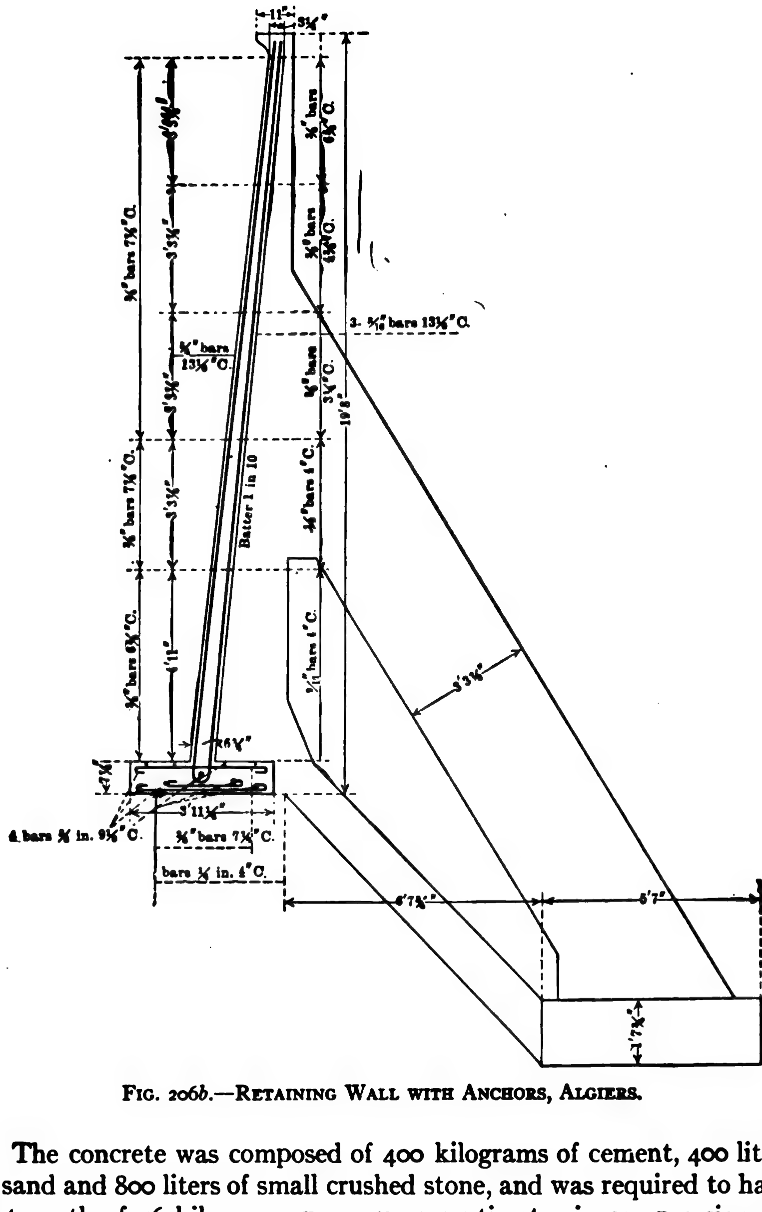


FIG. 206b.—RETAINING WALL WITH ANCHORS, ALGIERS.

The concrete was composed of 400 kilograms of cement, 400 liters of sand and 800 liters of small crushed stone, and was required to have a strength of 56 kilograms per square centimeter in compression, as required for such work by the ministerial circular of 1906. The

steel was to have an elastic limit of 25 kilograms per square millimeter.

The pressure on the foundation was fixed at 5 kilograms per square centimeter on the natural earth and 0.50 kilogram per square centimeter on the old fills. This last value must be used as taking account of the pressure resulting from the new fill, otherwise it would be unreliable.

The designers took the coefficient of equivalence m at 15, as provided in the ministerial instructions of Oct. 20, 1906. The pressure of the earth on the walls, which determines their dimensions, as well as the sides of the underway, has been considered to be equal to that of a liquid having a density four times less than the earth used for the fills, that is, of 1800 kilograms. This hypothesis is not justified in the specifications of the project; but it is easy to understand that it is equivalent to admitting that the coefficient of friction of the filling on itself is 0.75, neglecting the cohesion between the filling and the masonry. This coefficient of friction, 0.75, is entirely reasonable for earth containing a high proportion of rocks or stones, as in the filling used in this work.

The live load taken into account was for the walks, a uniform load of 400 kilograms per square meter, equivalent to a height of fill of 22 centimeters; for the roadway of 12 tons on each axle. The effect of these last have been calculated on a hypothesis somewhat inexact, but they are considered as being sufficiently exact for practical purposes.

The data given, amount on the one hand, to admitting that the effect of an isolated load is limited by the surface of a cone of revolution, generated by an inclined line, with a base of 1 and a height of 2, and on the other hand that the pressure is transmitted uniformly on the axis of the horizontal sections of the cone of revolution.

If, instead of an isolated load, we have several equal loads, A, A, A, A, such as the wheels of vehicles, the cones of pressure are replaced by four planes having a greater slope, with an inclination of 1 on the base to 2 vertical; two planes passing through the line A, A and the other two passing through perpendiculars erected through A and A in a horizontal plane. At a depth of h the load is distributed over a rectangle of which the base is $l+h$ and the height is h; l being the distance between the extreme loads A and A. However, when this rectangle meets a wall or buttress, the distributing surface is limited at that wall or buttress.

Passing from horizontal pressures to those acting on a vertical wall, there is applied in the first instance a coefficient of one-fourth

by analogy with the hypothesis which supposes the earth pressure to be equal to that of a liquid with one-fourth the density. The action of several isolated loads diminishes also with the depth, more rapidly than the depth h and less rapidly than the square of this dimension. This becomes very weak at the side, where the pressure of the earth is increased proportionately to h.

The effect of the rolling loads on the embankment covering the underway has been calculated with the empirical formula given in the instructions from the Conseil General des Ponts et Chausses, being given in a note inserted in the Annales of 1912, page 463.

When the designs of the retaining wall are examined, the difference between the size of the reinforcing in the vertical and inclined portions and that in the base, is immediately noted. This is because the diagonal braces rest upon and in the recent fill, subject to settlement; and they have been calculated to resist the weight of the fill resting upon them and the friction of the filling upon their lateral faces. For the bars in the bases resting on the old fill, they have not been considered as being subjected to these two kinds of stresses.

It would certainly have been of interest, from the point of view of reducing the cost, to place between the bars an economical masonry, composed of stones and grout, to resist the vertical bending of the upper piece, and the width of it could thus have been reduced.

The concrete work was constructed under the designing engineers, and the entire work carried out according to their plans, which have been described. The general direction of the works was under Maitre-Devalon, engineer; Mazet, the commandant of mobilization in Quest d'Alger, and Salle, the officer on charge for the Ponts et Chaussees.

Retaining Wall Shore Protection

The reinforced concrete retaining wall, which was built as a sea-wall shore protection at Allenhurst, N. J., from the plans of M. H. Lewis, consulting engineer, Fig. 207, presents some novel features in light wall construction. The calculations were made on the basis of the pressure from the 5 feet wedge of wet filling between the old timber bulkhead and the new wall, but properly they should have disregarded the old timber work, which will soon rot out and require the new wall to carry the entire thrust as usually computed.

While the wall is only 3 feet thick at the bottom and 1 foot 3 inches thick at the top, the fact that it is anchored by the sidewalk slab to the curb, and is provided with a 10-foot base of 3 feet minimum

thickness, probably makes it secure from overturning in any event. The pile foundation is evidently nothing more than an anchor for the base, and should a similar design be employed elsewhere not less than three rows of piles should be used unless the foundation is of the very best, and unless the footing course is carried deeper.

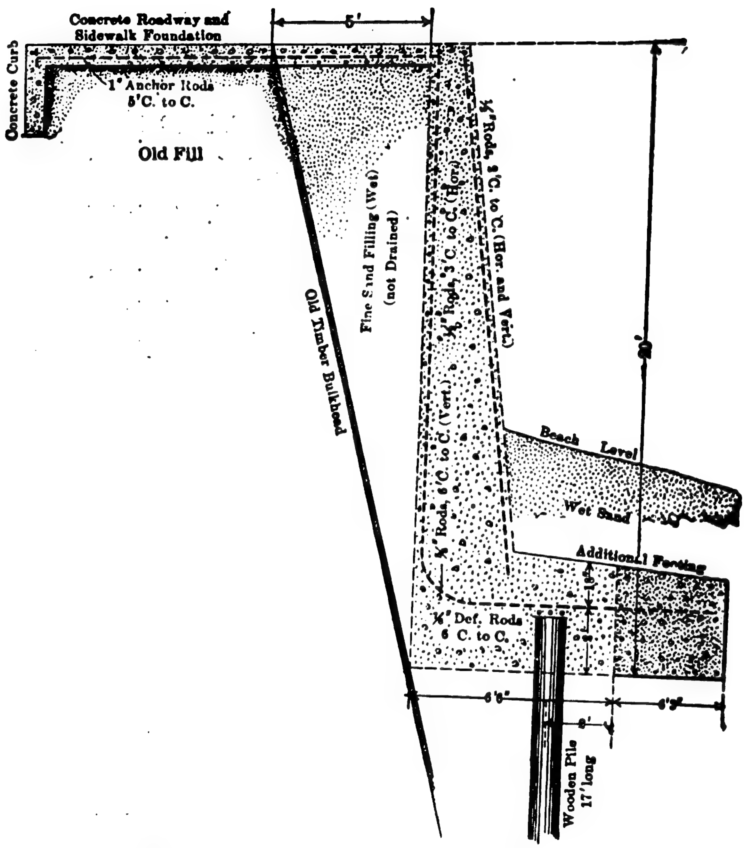


FIG. 207.—SHORE PROTECTION, ALLENHURST, N. J.

The cost of such a wall, 20 feet high, at ordinary prices of material, would be about 25 per cent greater than a wall of steel sheet piling anchored with rods, and protected with a cement gun facing on wire netting.

Gibb's Practical Retaining Wall Design

The object of the following investigation by H. M. Gibb is to give a method of designing reinforced-concrete retaining walls which

is quick, accurate and simple to apply to all common types and which gives results within about 10 per cent of the rigid analysis; also to show a method of making standard designs in which all moments and shears and stresses on the foundation and reinforced slabs of the wall are given in terms of the height of the fill.

The following is the notation used:

ϵ = angle made by top of fill with the horizontal;
ϕ = angle of repose of fill;
a = angle which the resultant of all forces on the wall makes with the vertical;
h = distance from underside of base to top of fill (see Figs. 209 and 210;
d = distance from top of base to top of fill;
b = width of base of wall;
x = distance of face of vertical stem of wall from toe;
w = weight of fill in pounds per cubic foot;
p = weight of equivalent fluid in pounds per cubic foot;
W = resultant vertical pressure;
P = resultant fluid pressure;
s = weight of surcharge in pounds per square foot;
h' = depth of fill equivalent to s;
k = load per lineal foot on span;
$B.M.$ = bending moment;
l = distance c. to c. of counterforts;

$$C=\left[\cos^2 \epsilon \frac{\cos \epsilon-\sqrt{\cos^2 \epsilon-\cos^2 \phi}}{\cos \epsilon+\sqrt{\cos^2 \epsilon-\cos^2 \phi}}\right].$$

The loads on a retaining wall are, first, the vertical loads which include the weight of the wall, and the weight of the fill and the surcharge resting on the heel slab; and second, the horizontal thrust of the fill on the vertical stem of the wall.

The action of the fill on the wall is best realized by considering the fill as a semifluid which exerts a vertical pressure due to its own weight, and a horizontal fluid pressure due to its semifluid action, i.e., if y is the distance below the top of the fill then the vertical intensity of pressure at this point $=wy$ and the horizontal intensity of pressure $=py$ (see Fig. 209).

To compute the value of p, Rankine's general formula for a vertical wall is used (Fig. 208) in which it is shown that $p=wC$. The graph gives values of C for all values of ϕ and ϵ, values of ϕ given

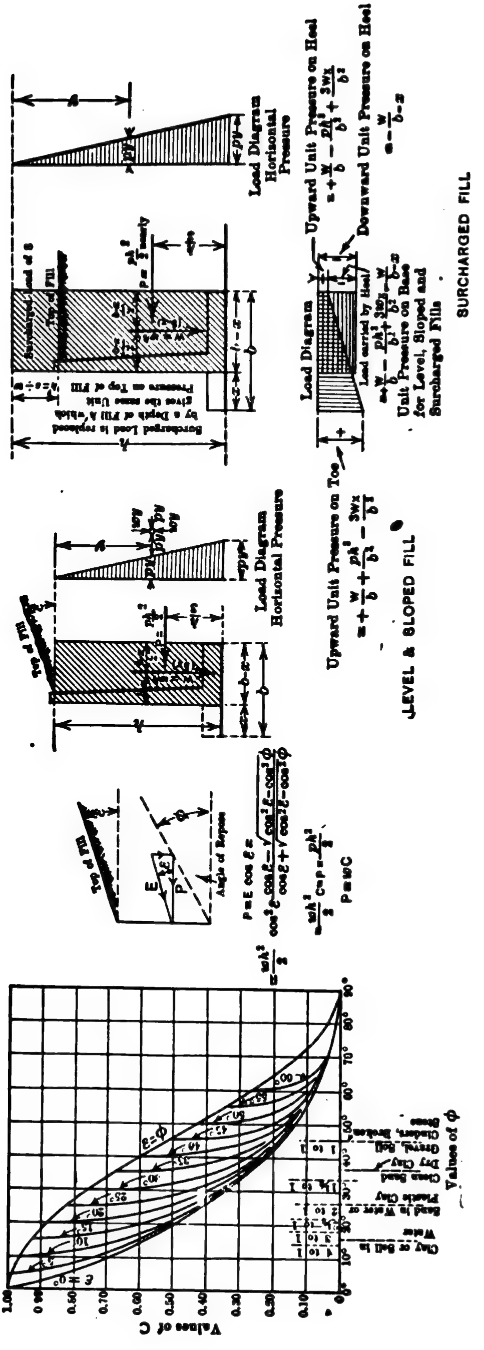


FIG. 208.—ANGLE OF REPOSE DIAGRAM. FIG. 209.—LEVEL AND SLOPED FILL. FIG. 210.—SURCHARGED FILL.

for different materials are taken from the American Civil Engineers Pocket Book. Useful slopes have also been shown.

Fig. 209 gives the basis on which the analysis of the wall is made, the weight of the wall and the backfill combined is taken as being represented by the shaded area and of the same weight as the fill, i.e., the shaded part of the wall is taken as the same unit weight as the fill and the weight of the unshaded parts of the wall and fill are neglected. The horizontal pressure is that due to a fluid of p pounds per cubic foot.

Fig. 210 gives the case of the surcharged wall, and it is to be noted that h' is not the height of the surcharge but the height of fill which gives the same intensity of loading as the surcharge. In this case

$$P=p\left(\frac{h^2-h'^2}{2}\right),$$

but $P=\frac{ph^2}{2}$ is used as it only involves an error of 4 per cent when h' is 20 per cent of h, which is a heavy surcharge.

There is also given on Figs. 209 and 210 a load diagram of pressures on the base; the pressure diagram of the loads on the foundation is the whole of the vertically lined area, the whole of the horizontally lined area is the load on the heel slab, the cross-lined area represents balanced pressures, and the remaining vertically and horizontally lined areas represent the loads to be resisted by the toe and heel slabs respectively.

The friction of the fill on the back of the wall, and the vertical component of E are neglected.

The conditions for stability are:

1. The resultant must cut the base within the middle third.

2. The resultant must not make an angle with the normal to the base of the wall greater than the angle of friction.

By taking moments about the outside edge of the middle third and equating

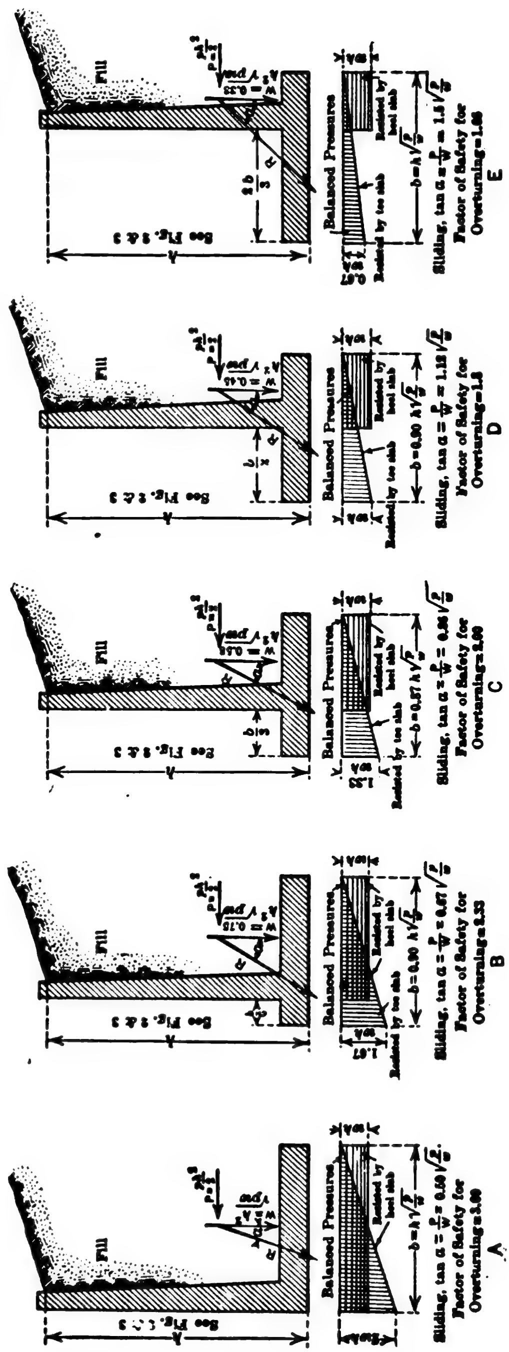


FIG. 211.—STANDARD TYPE, CANTILEVER WALL.

them to o, we will get a wall which satisfies the first condition, and on which the maximum toe pressure is twice the average, and the minimum heel pressure is zero.

There is a certain position of the vertical slab on the base slab which will give the minimum length of base, this occurs when $x=\frac{b}{3}$ but as it is not always possible or advisable to build every wall to this proportion, the results of the analysis of Types A to E (the position of wall on base being varied) are given on Fig. 211.

The analysis of Type C is given here in full, the remaining analysis are similar.

$$W=\frac{2whb}{3}, \qquad P=\frac{ph^2}{2}.$$

Taking moments round outside edge of middle third

$$\frac{2whb^2}{9}=\frac{ph^3}{6},$$

from which

$$b=0.87h\sqrt{\frac{p}{w}},$$

substituting this value of b, $W=0.58wh^2\sqrt{\frac{p}{w}}$.

Average unit pressure on base $=\frac{W}{b}=0.66wh$.

Maximum pressure on toe $=1.33\ wh$.

Minimum pressure on heel $=0$.

Resistance to overturning is obtained by taking moments around the toe, then

$$\frac{\text{Negative moments}}{\text{Positive moments}}=\frac{\left(0.58wh^2\sqrt{\frac{p}{w}}\right)\times\left(0.58h\sqrt{\frac{p}{w}}\right)}{\frac{ph^3}{6}}=2.00.$$

To find if wall is safe against sliding

$$\text{Tan}\ \alpha=\frac{P}{W}=0.86\sqrt{\frac{p}{w}}.$$

On Fig. 211 five types (A, B, C, D, E) of retaining walls are given with the magnitude of the unit pressures, ratio of width of base to height of fill, resistance to overturning and values of tan α.

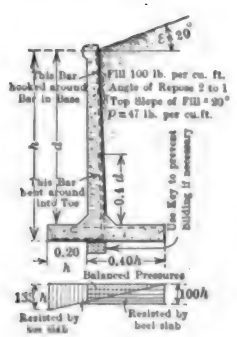
The following principles are noted:

The length of the base varies as $\sqrt{\frac{p}{w}}$.

As the vertical stem is moved back from the toe toward the heel, the unit toe pressures decrease in intensity, the length of the base b decreases till it is a minimum when the face of the stem is $\frac{b}{3}$ from the toe, after this it increases in value, but between positions of $\frac{b}{6}$ and $\frac{b}{2}$ from toe, the difference amounts to only 3%.

3. The resistance to sliding decreases as the vertical stem is moved back from the toe toward the heel.

4. The resistance to overturning decreases as the stem is moved back from the toe toward the heel.



Moments and Shears are for Slabs 12" wide.

Vertical Slab.

Max. B.M. $=7.85d^3$.
Max. Shear $=23.5d^2$.
For Moments and Shears at any Depth y, use y for d in above.

Toe Slab.

Max. B.M. $=2.36h^3$.
Max. Shear $=22.1h^2$.

Heel Slab.

Max. B.M. $=3.50h^3$.
Max. Shear $=20.0h^2$.

h and d in Feet.
B.M. in Foot-lb.
Shear in lb.
$\tan\alpha=0.59$, $\alpha=30°$
Factor of Safety for Overturning $=2.00$

FIG. 212.—SLIDING FORCE ON WALL.

If the above wall Type C is wanted with a factor of safety against overturning of 3, then by taking moments round the toe and equating the moment of W to three times the moment of P we will get $b=1.05h\sqrt{\frac{p}{w}}$.

From equations on Figs. 209 and 210 the unit pressures on the base at toe and heel willl be found to be $+0.92wh$ and $+0.42wh$, respectively.

If it is required in Type C to find the length of base necessary to prevent sliding, then

$$\tan\alpha=\frac{P}{W}=\frac{\frac{ph^2}{2}}{\frac{2}{3}whb},$$

and
$$b = \frac{0.75ph}{w \tan \alpha}.$$

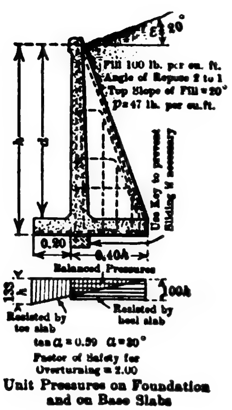
After substituting the values of p, w and $\tan \alpha$ in this equation, the solution of the pressures on the base can be made by the equations on Figs. 209 and 210.

Design of Standards.

Where a large number of walls retaining similar fill but of varying height are to be designed, the standardization can be carried further than shown on Fig. 211, and in Figs. 212 and 213 are given standard cantilever and counterfort walls based on Type C.

The fill designed for weighs 100 pounds per cubic foot, and slopes top at an angle of 20°, and has an angle of repose of 2 to 1. From Fig. 208, $C=0.47$ and $p=47$ pounds per cubic foot.

In Figs. 212 and 213 are shown a cantilever and a counterfort wall with all moments and shears and unit pressures in toe, heel,



Moments and Shears are for Slabs 12″ wide.

Vertical Slab.

Max. B.M. at Depth $d = 4dl^2$, if $M = Kl^2/12$.
Max. B.M. at Depth $d = 6dl^2$, if $M = Kl^2/8$.
Max. Shear at Depth $d = 23.5dl$.
For Moments and Shears at any Depth y, use y for d in above.

Toe Slab.

Max. B.M. $= 2.36h^3$.
Max. Shear $= 22.1h^2$.

Heel Slab.

Max. B.M. $= 8.33hl^2$, if $M = Kl^2/12$.
Max. B.M. $= 12.5hl^2$, if $M = Kl^2/8$.
Max. Shear $= 50hl$.
Moments and Shears decrease towards Toe.

Stress resisted by Steel in Back of Counterfort $= 23.5d^2l$.
Shear in Counterfort $= 23.5d^2l$.

Sufficient Reinforcement to be provided between Vertical Slab and Counterfort, and Heel Slab and Counterfort to resist Shear of Vertical Slab and Heel Slab respectively.
h, d, l, and y in Feet.
B.M. in Foot-lb.
l = C. to C. of Counterforts. Shear in lb.

FIG. 213.—STANDARD TYPE, COUNTERFORT WALL.

and vertical slabs expressed in terms of the height of the fill, these values are arrived at by substituting the values of p and w in Type C, Fig. 211, and calculating the moments and shears by the usual methods.

To find the stress in the back of the counterfort the lever arm of the beam is for this type $\frac{d}{3}$ approximately. Then

$$\text{Stress} = \frac{pd^3l}{b} \div \frac{d}{3} = 23.5d^2l.$$

The value of tan α is 0.59, and if this is too high for the material on which the wall is founded, it will be necessary to introduce a key in the base, as shown in dots, to give greater resistance to sliding.

Fig. 214 is a graph of the numerical coefficients in equations on Fig. 211 for different values of $\frac{x}{b}$ and by means of this graph all intermediate values of these coefficients can be arrived at.

In conclusion it may be stated that in the cantilever wall, all three slabs (vertical, heel and toe slabs) of the wall are cantilevers, with the support at their junction; the toe slab is reinforced in the bottom, the heel slab in the top, and the vertical slab in the back.

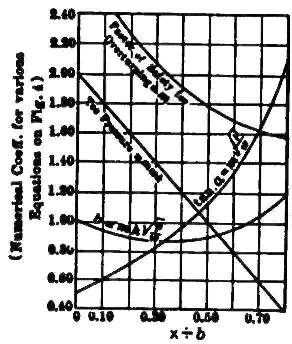


FIG. 214.—WALL COEFFICIENT DIAGRAM.

In the counterfort wall, the vertical slab and the heel slab are beams spanning between the counterforts, which are their supports, the vertical slab is reinforced in the front, and the heel slab on the bottom, and with short lengths of bar over the counterforts to take negative moments; the toe slab is a cantilever, reinforced on the bottom.

In all cantilever slabs special attention should be taken to insure that bars are given sufficient bond over the supports to develop their full strength.

The above is taken from Engineering News, July 24, 1913.

Reinforced Surcharged Walls

The walls constructed for the Canadian Northern Ry. terminal at Montreal, Canada, were described in the Journal of the Engineering Institute of Canada for April, 1919. They are of the cantilever surcharged type, Fig. 215, and were built primarily as retaining walls, but also carry the walls of buildings. The adjoining expensive property made necessary the use of cantilever walls although ordinarily counterfort walls would have been the cheapest.

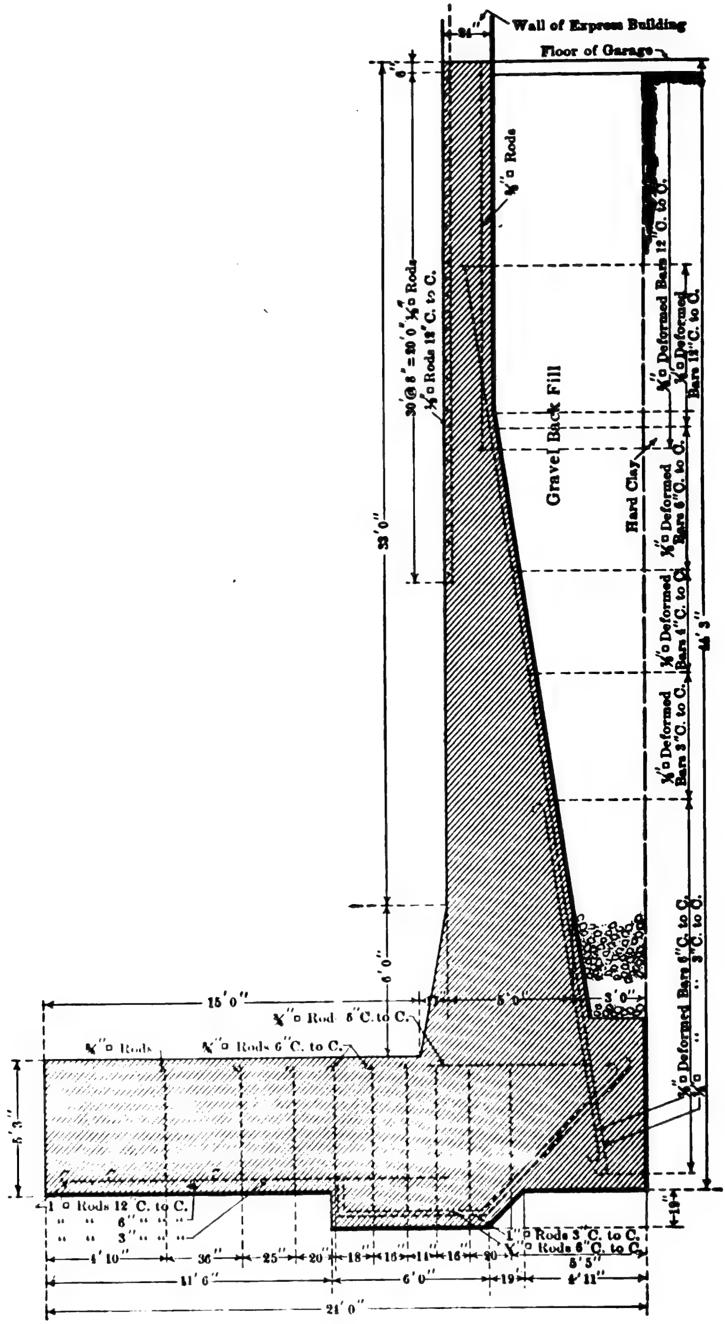


FIG. 215.—CANADIAN NOR. RY. WALL, MONTREAL.

The walls are all calculated for stability without the vertical load from the buildings, and this tends to decrease the pressure at the toe, as well as the stresses in the reinforcing. The surcharge was assumed at 200 pounds per square foot of the area of the building. The ground back of the walls is very hard and stood up nearly vertical, although the angle of repose was taken 30 degrees to guard against possible slips and hydraulic pressure behind. Great care was taken, however, with the drainage. The foundation bed was also hard pan.

The total amount of concrete was 2256 cubic yards, and there was 143,300 pounds of reinforcing used, or 64 pounds per cubic yard. The walls were designed by J. C. Krumm, and constructed under the direction of C. C. Briggs, Supervisor of Buildings for the Canadian Northern Railway.

Comparative Cost of Solid and Reinforced Walls

The retaining walls of reinforced concrete used by the Great Northern Railway, to protect the approaches to the tunnel under the city of Seattle, according to data furnished by O. S. Bowen, Asst. Ch. Eng., were compared with walls of gravity section as shown in Fig. 216, the width of base being taken at 0.4 the height and a face batter of 1 in 12.

The percentage of saving was found to be 20.4 per cent for walls 10 feet high; 36.4 per cent for walls 20 feet high; 43.3 per cent for walls 30 feet high; and 45 per cent for walls 40 feet high. This comparison was made by including the steel reinforcing cost in terms of concrete at $6 per cubic yard. The concrete in the reinforced and plain walls was taken at the same cost per cubic yard, whereas the cost of the reinforced walls should usually be taken at a considerably higher rate, thus reducing the percentage of saving by a very considerable amount, but still showing a decided difference in favor of the reinforced walls. The spacing of the ribs or buttresses was taken at 7 feet 6 inches for the 30 and 40 feet high walls; at 10 feet for the 20 feet high walls; and at 15 feet for the 10-feet high wall. Should a different spacing be adopted for another piece of work then the percentage of saving would be changed from the figures given.

The design itself is one that may be found very good for other locations, as it is plain, extremely simple and of good proportions. The base is of sufficient width for any moderately solid foundation bed, and the only suggestion would be to provide proper drainage

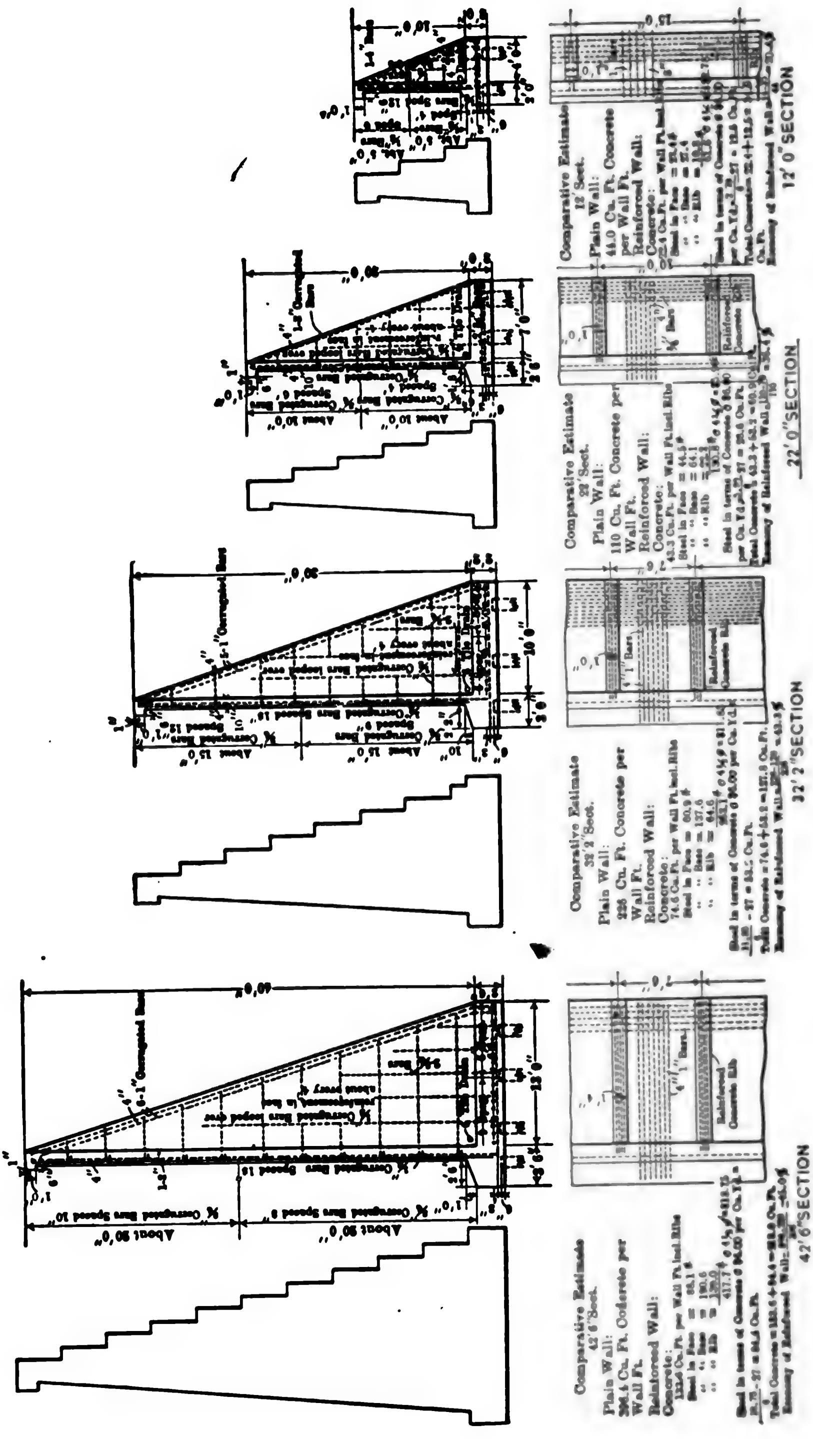


FIG. 216.—GREAT NORTHERN R.R. RETAINING WALLS.

and weep-holes, which of course are not shown on this plan, which was made for the purpose only of comparing the cost.

TABLE XXXIII.—CONTENTS OF SOLID RETAINING WALLS

Height.	Cu. Yds. per Lin. Ft.	Height.	Cu. Yds. per Lin. Ft.	Height.	Cu. Yds. per Lin. Ft.
10	1.66	20	4.07	30	8.32
12	2.04	22	4.62	32	9.25
14	2.41	24	5.56	34	10.36
16	2.96	26	6.48	36	11.45
18	3.52	28	7.40	38	13.35

Base =0.4 height. Face batter 1 to 12.

TABLE XXXIV.—CONTENTS REINFORCED RETAINING WALLS

Height, Feet.	Cu. Yds. per Lin. Ft.	Reinforcing.		Height, Feet.	Cu. Yds. per Lin. Ft.	Reinforcing.	
		Lbs. Cu. Yd.	Lbs. Lin. Ft.			Lbs. Cu. Yd.	Lbs. Lin. Ft.
10	0.93	64.5	60.0	26	2.23	91.8	205.0
12	1.04	71.2	74.0	28	2.49	94.2	235.0
14	1.19	74.0	88.0	30	2.78	95.2	265.0
16	1.34	76.1	102.0	32	3.18	92.8	295.0
18	1.49	81.2	121.0	34	3.63	90.5	328.0
20	1.67	83.9	140.0	36	4.08	88.8	361.0
22	1.86	86.0	160.0	38	4.53	86.8	394.0
24	2.04	89.1	182.0	40	5.00	85.4	427.0

Relative Economy of Cantilever and Counterfort Walls

The relative economy of cantilever and counterfort reinforced concrete retaining walls, and conditions for the economical design of reinforced concrete retaining walls is discussed in an article by George Paaswell, in *Engineering and Contracting* for February 26, 1919.

While, clearly, for some given height of wall, a counterforted wall of reinforced concrete becomes cheaper than an L or T shaped concrete wall, a search of pertinent literature fails to yield any simple method of obtaining such a height, save by actual comparison of two completed designs. It may be well worth while, then, to establish some method of getting this "critical" height.

It is true, extraneous factors may control the selection of types of walls and their component dimensions, but, generally, a wall is so designed as to satisfy, most economically, its stresses.

Again, the bending moment, shear of adhesion stress may, each in turn, control the necessary thickness of the several parts of the wall as the height varies. It is to be noted, however, that with rare exception, such several stresses usually require about the same thickness of section, though, probably, a greater variation in the amount of reinforcement required. In assuming, then, that the wall dimensions

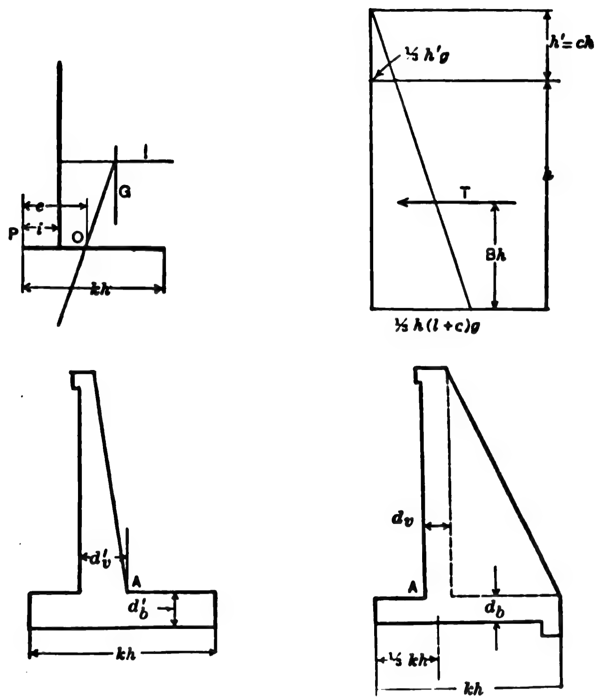


FIG. 217.—RELATIVE ECONOMY OF RETAINING WALLS.

follow the theoretical requirements, a large percentage of actual cases are covered and, if these dimensions are taken in accordance with the stress of simplest expression, no serious error results. With this in mind, the various thicknesses of both the cantilever and the counterforted are those given by the bending moment requirements.

Before entering into the comparative economy of the two general types of walls, it may prove serviceable to establish some general properties governing all types of reinforced concrete walls.

By ignoring the difference in weight between the masonry and the earth, the ratio between the width of base and the height of wall may be expressed quite simply as follows: Using Rankine's expression

for the thrust with a horizontal upper surface and with the angle of repose 30°, together with a surcharge of depth h' the thrust is found to be

$$T = g\frac{h^2}{6}(1+2c), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where c is the ratio $h' : h$.

The thrust is located at the center of gravity of the applied loading, a distance Bh above the base, where B is found to be

$$B = \frac{1}{3}\,\frac{1+3c}{1+c}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The weight G is $w(1-i)h(1+c)g$, and the lever arms about P and O are respectively,

$$\frac{w(1+i)}{2}, \quad w\left(\frac{1+i}{2}-e\right).$$

Let $w = kh$.

Taking moments about P and O and eliminating the irrelevant factors, k is finally found to be

$$k = \frac{1}{3}\sqrt{\frac{1+3c}{1+c}}\sqrt{\frac{1}{(1-e)^2-(i-e)^2}}. \quad . \quad . \quad . \quad . \quad (3)$$

Clearly, the least value of k is given when the denominator of the second radical is a maximum, or when $(i-e)^2$ vanishes, that is, when $i=e$. Thus the most economic section is to be had when the location of the vertical arm is just over the point of application of the resultant.

In the work that follows, since it is a comparative estimate of the cost of the two types of walls that is sought, it is justifiable to select as a type for the present analysis, that giving the simplest expression. It is quite clear that variations in the toe length, or in the assumed position of the resultant, will not affect, to any material extent, the comparative estimate. For this reason the last established general condition of economy, $i=e$, is adopted, with a further provision, that $e=\frac{1}{3}$—which is the usual goal in retaining wall design. With these conditions equation (3) becomes

$$k = \frac{1}{2}\sqrt{\frac{1+3c}{1+c}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

In a reinforced concrete section the resisting moment is given by

$$M = Kbd^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

K is a constant depending upon the unit compressive strength and the percentggc of steel adopted. For an economic section, the percentage of steel is about 0.75 per cent, and since $K = 0.5kjf_c$ (the nomenclature of the Special Concrete Committee of the Am. Soc. C. E. is used here) with this percentage of steel $0.5kj = 0.17$, and if a value of f_c of 650 pounds per square inch is used, and then expressed in pounds per square foot, $K = 16{,}000$. Using $b = 1$, M is then

$$M = 16{,}000d^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The bending moment due to the thrust is TBh, and replacing by the values previously found becomes

$$M = \tfrac{1}{18}g(1+3c)h^3. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

With g the usual value 100 pounds per cubic foot and equating equations (6) and (7),

$$d_v = 0.0186h^{3/2}\sqrt{1+3c} = C_v h^{3/2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

In getting the moment for the footing, using the value of k as given in equation (4), it is found that the footing moment is about 0.7* of the arm moment as found in equation (7), and the required thickness of the base slab is

$$d_b = \sqrt{.7}\,d_v = 0.84d_v. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

In the counterfort walls, if m is the distance between the counterforts and the moment is taken as $\frac{Wm^2}{12}$ (i.e., the slabs are assumed continuous), then since

$$W = \frac{g}{3}h\,(1+c),$$

the moment becomes

$$M = \frac{gh(1+c)m^2}{36}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

* Exactly the base moment is

$$M = I\frac{gh^3(1+3c)}{18}$$

where $I = (1-i)^2(1+2i)$ when $e = \frac{1}{3}$.

Adopting the same concrete constants as before and equating this moment to the resisting moment,

$$d'_v = 0.0132m\sqrt{h(1+c)} = C_v'm\sqrt{h}. \quad . \quad . \quad . \quad . \quad (11)$$

At the outer edge of the footing, since the soil intensity is zero, the weight on the slab is that of the superimposed earth or

$$W = gh(1+c).$$

The moment is then

$$\frac{gh(1+c)}{12}m^2,$$

or three times the corresponding vertical slab moment, whence

$$d'_b = d'_v\sqrt{3}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

The counterfort is usually about 1 foot thick and will so be taken here.

The cost of the steel rods is a small part of the total cost of the wall and the relative difference in cost of steel rods between the two wall types would thus be negligible.

The amount of face forms for the vertical arms of either type is the same and is thus ignored. The variable factors of importance in the two types of walls are then, the concrete yardages and the face forms of the counterforts.

Let L be the total length of wall taken, r be the cost per cubic foot of placing the concrete into the wall (the cost of placing concrete is substantially the same for both types) and finally t be the cost of the forms per square foot. For the counterforted wall, the amount of concrete is

$$L(d'_v h + khd'_b) + \frac{L}{m}\frac{kh^2}{2},$$

and the total cost of the concrete is then

$$Lrh\left\{d'_v(1+k\sqrt{3}) + \frac{hk}{2m}\right\}. \quad . \quad . \quad . \quad . \quad (13)$$

The cost of the face forms of the counterfort is

$$\frac{1}{L}\frac{L}{m}\frac{kh^2}{2}2$$

making the total cost of this wall (meaning the total cost under investigation)

$$Lrh\left\{d'_v(1+k\sqrt{3})+\frac{hk}{2m}\left(1+2\frac{t}{r}\right)\right\}. \quad . \quad . \quad . \quad (14)$$

In the T cantilever wall, the width at the coping is taken as 1 foot and the back battered unbrokenly to the required thickness at the base. The volume of this wall is

$$L\frac{1+d_v}{2}h+khd_b=Lh[\tfrac{1}{2}+d_v(\tfrac{1}{2}+0.84k)],$$

and the total cost of the wall is thus

$$Lhr[.5+d_v(.5+.84k)]. \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

Equating (14) and (15)

$$d'_v(1+k\sqrt{3})+\frac{hk}{2m}\left(1+2\frac{t}{r}\right)=.5+d_v(.5+.84k).$$

Replacing the thicknesses of the sections by their values as found above

$$C'_v m\sqrt{h}(1+k\sqrt{3})+\frac{hk}{2m}\left(1+2\frac{t}{r}\right)=.5+C_v h^{3/2}(.5+.84k).$$

Later it will be shown that the economic spacing of counterforts is given by $m=3.1Rh^{1/4}$, where $R=\sqrt{1+2\frac{t}{r}}$. Using this expression, there results a quadratic in $h^{3/4}$,

$$C_2 h^{3/2}-RC_1 h^{3/4}+.5=0,$$

where

$$C_2=.0132\sqrt{1+c}\,3.1(1+k\sqrt{3})+\frac{k}{6.2}$$

$$C_1=.0186\sqrt{1+3c}(.5+0.84k)$$

giving a value of $h^{3/4}$,

$$h^{3/4}=\frac{RC_4+\sqrt{R^2C_1^2-2C_2}}{2C_2}.$$

Table A gives a series of values of h for several values of the ratio t/r and of the ratio h'/h or c.

TABLE A.—CRITICAL HEIGHTS OF WALLS

t/r	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{4}$	1
c	15	22	28	33
0	15	22	28	33
$\frac{1}{4}$	11	17	22	27
$\frac{1}{2}$	10	15	19	23

To determine the spacing of counterforts to give the most economic wall sections, it is seen that equation (14) is the required expression for the variable cost as the position of the counterfort changes.

If, by the theory of Minima and Maxima, the derivative of this expression with respect to m is put equal to zero, there results, after again replacing the several thicknesses by their values as previously found:

$$m=\sqrt{\frac{k\sqrt{h}\left(1+2\frac{t}{r}\right)}{2C'_s(1+k\sqrt{3})}}+\sqrt{\frac{\sqrt{h}}{2}\left(1+2\frac{t}{r}\right)\frac{k}{.0132\sqrt{1+c}(1+k\sqrt{3})}}.$$

Putting $R=\sqrt{1+2\frac{r}{t}}$, and noting that the expression after using the value of k as given in equation (4),

$$\sqrt{\frac{k}{(1+k\sqrt{3})\sqrt{1+c}}}$$

is practically constant and equal to one-half,

$$m=3.1Rh.$$

Table B gives a series of values of m for several values of the ratio t/r and for a range of heights.

TABLE B.—ECONOMICAL SPACING OF COUNTERFORTS

h t/r	15	20	25	30	35	40	50
$\frac{1}{4}$	7.5	8.1	8.6	8.9	9.3	9.6	10.2
$\frac{1}{2}$	8.6	9.3	9.8	10.2	10.7	11.0	11.6
$\frac{3}{4}$	9.6	10.4	11.0	11.5	12.0	12.4	13.1
1	10.6	11.4	12.0	12.6	13.1	13.5	14.3

It is reasonable to expect that the laws governing the theory of probabilities hold here and thus that the small errors introduced in the above approximations are compensatory. By several trials

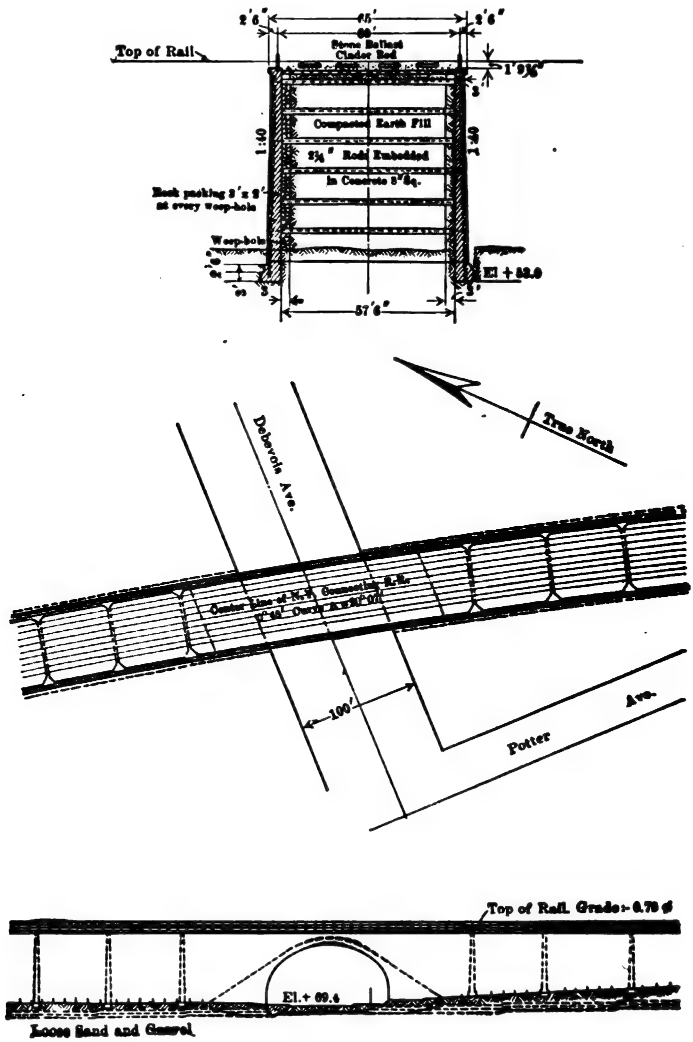


FIG. 218.—BOX RETAINING WALL, HELL GATE ARCH, EASTERN VIADUCT.

and comparisons of actual wall sections it is found that the tables furnish quite an exact clue to both the " critical height " *h* and the economic spacing of the counterforts.

Box Retaining Wall. Hell Gate Bridge

The eastern approach to Hell Gate arch, designed by Gustav Lindenthal, Consulting Engineer, has a total length of 3550 feet, and consists mostly of reinforced concrete arches over the streets, which form a monolithic structure with the novel type of retaining walls of the embankment.

The embankment consists of two longitudinal retaining walls, connected and held in relative position by horizontal tie rods, which are imbedded individually in a shell of concrete for protection against corrosion. These rods resist the pressure from the earth fill. (Fig. 218.) For additional stability, the two walls are connected by thin cross walls, about 5 feet apart. The arches consist of a comparatively thin barrel, reinforced by vertical ribs. The fill is mixed clay, sand, and gravel, carefully placed in 12-inch crowned layers, and thoroughly tamped, so as to form a uniform compact mass which exerts a comparatively small pressure on the retaining walls. It is thoroughly drained by chimneys of rock packing which extend along the walls from the top of the fill to the weep-holes at the bottom.

The walls and arches have perfectly plain surfaces and a simple coping. No attempt has been made at architectural treatment, because the territory in the vicinity is being built up mostly by industrial buildings which hide that portion of the railroad from prominent view. This embankment construction is considerably cheaper than the ordinary type, which consists of a fill between two independent gravity walls. For a height of 50 feet the latter type would have cost from 30 to 40 per cent more. The above account is taken from the account by O. H. Ammann, Prin. Asst. Engr. of the structure, in the 1917 transactions of the American Society of Civil Engineers.

Concrete Block and Cellular Retaining Walls

The track elevation at Milwaukee for the Chicago, Milwakuee and St. Paul Ry. developed very poor foundation conditions, and Chas. F. Loweth, Chief Engineer, found it necessary to employ two new types of retaining wall, which were described as follows, in *Engineering News*, May, 1915. Both of the designs were cheaper than any requiring piles, and both involved the ignoring of the usual degree of security, but gave a sufficient factor of safety against failure to warrant their adoption.

The design shown in Fig. 219 is an adaptation of the dry stone wall, and consists of concrete blocks placed one upon the other with horizontal beds, each course being offset from the course below so as to give the wall an appreciable batter, as shown in the cross-section. In order to give resistance to sliding, the blocks were made with

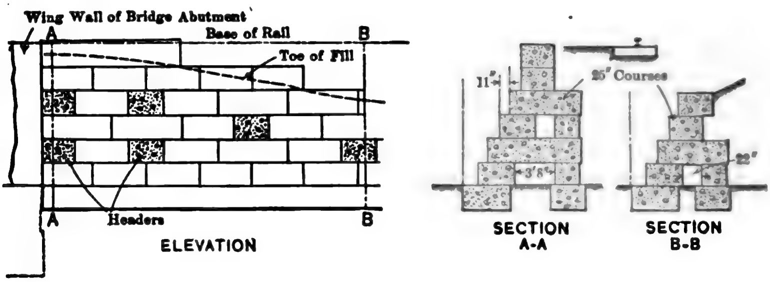


FIG. 219.—CONCRETE BLOCK RETAINING WALL, C. M. & ST. P. RY.

corrugated upper and lower faces, by casting them in forms lined with corrugated iron.

Ribs extending back into the embankment were provided at intervals of about 9 feet, composed of headers interspaced with small blocks. These ribs simply served to support the wall until the embankment was placed, and were not intended to add to the stability against lateral earth pressure. One wall of this type was built, but further use was discontinued in favor of another type which proved superior in several respects.

The other type was a reinforced concrete cellular retaining wall, based on the idea of the timber crib, adapting it to monolithic concrete construction rather than the unit type of wall, as has been done in some cases. The method consists of a series of rectangular bottomless boxes of reinforced concrete, as shown in Fig. 220. The wall was analyzed according to the ordinary accepted methods, except that it was assumed that the passive resistance against the front face of the rear wall would exactly counterbalance the active earth pressure on the rear face. The resistance to the overturning moment of the active earth pressure against the front wall is afforded by the weight of the entire wall and that of the earth superimposed on the back and side walls. The distance between the front and back walls as shown in the design was varied by trial until a section was obtained which gave the desired pressure conditions, namely, that the bearing pressure would be practically uniform under the entire base of the section for the assumed condition of loading. For the greater heights it

was found that, instead of carrying the rear wall back the required distance, it was more economical to increase the top width so that it would carry a greater earth load, as shown in the lower section. The four walls making up the cell are designed to take the loads applied to them, according to the accepted reinforced concrete prac-

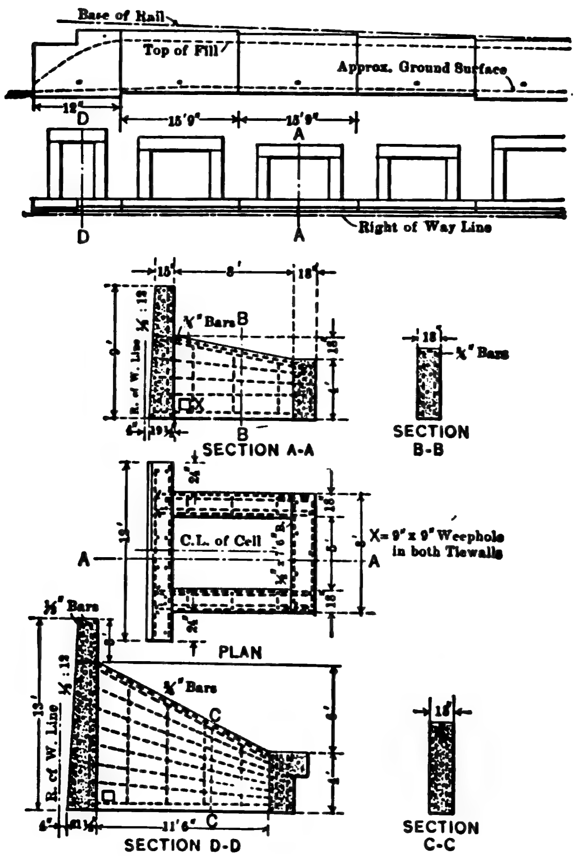


FIG. 220.—CELLULAR CONCRETE RETAINING WALL, C. M. & ST. P. RY.

tice. The stress in the front wall is reduced by spacing the side or tie walls in such a manner that the front wall acts as an overhanging slab. On the other hand it was found undesirable to extend the rear wall beyond the limits of the tie walls. As it was anticipated that these walls would settle materially and also tip either forward or backward to some extent, an allowance of from 9 to 15 inches was made in the height of the wall and the front face was set back 4 inches from the

property line, and battered to such an extent that the expected settlement would not result in encroachment upon adjoining property.

These walls depend largely upon the weight of the embankment for their stability and it was expected that they would settle with the embankment. Consequently, footings were considered unnecessary and the ground was simply leveled off to an even bed for the erection of the forms for the neatwork. The sections were made 16 feet to permit the use of stock lengths of lumber. Ordinary practice in form work was followed for the front face, but the forms for the rear face of the front wall, the tie and rear walls were made as rough and as cheap as possible without endangering the quality of the concrete from leaky forms. The joints in the front wall, between the adjoining cribs, were made plain without any keys or dowels, so that unequal settlement of adjoining sections would not result in local injury.

Weep-holes were provided in the center of each cell and in the joints between cells. It was thought necessary to provide drainage in this manner, as the anticipated settlement of the wall would result in the destruction of any line of drain pipes placed behind; and with the 4-inch space between the property line and the face of the wall, drainage can be provided in front, if the leakage from the weep-holes is objectionable. About 1700 feet of the cell type was required in the truck elevation project, and in general the walls were used in the embankment slope at some distance from the track. The greatest height of wall is about 14 feet where the track was elevated about 17 feet above the original ground surface. In one case a wall 12 feet 3 inches high was used, with the center line of track 8 feet 6 inches from the front face of the wall.

Pennsylvania Railroad Retaining Walls

The standard retaining walls of the Pennsylvania Railroad, Alex. C. Shand, Ch. Eng., are shown in Fig. 221, the thickness of the base for ordinary walls without surcharge or when no tracks are to be carried being 0.35 of the height above the ground line, and 3 feet in thickness at the top under the coping. When an ordinary surcharge or else railway tracks are to be carried at a distance of not less than 6 feet 3 inches from the face of the wall at the top, then the thickness of the base is made 0.40 of the height above the ground line, and 3 feet thick at the top under the coping. The wall when of concrete to be made in sections of about 30 feet between expansion joints, which are to be made as shown on the drawing;

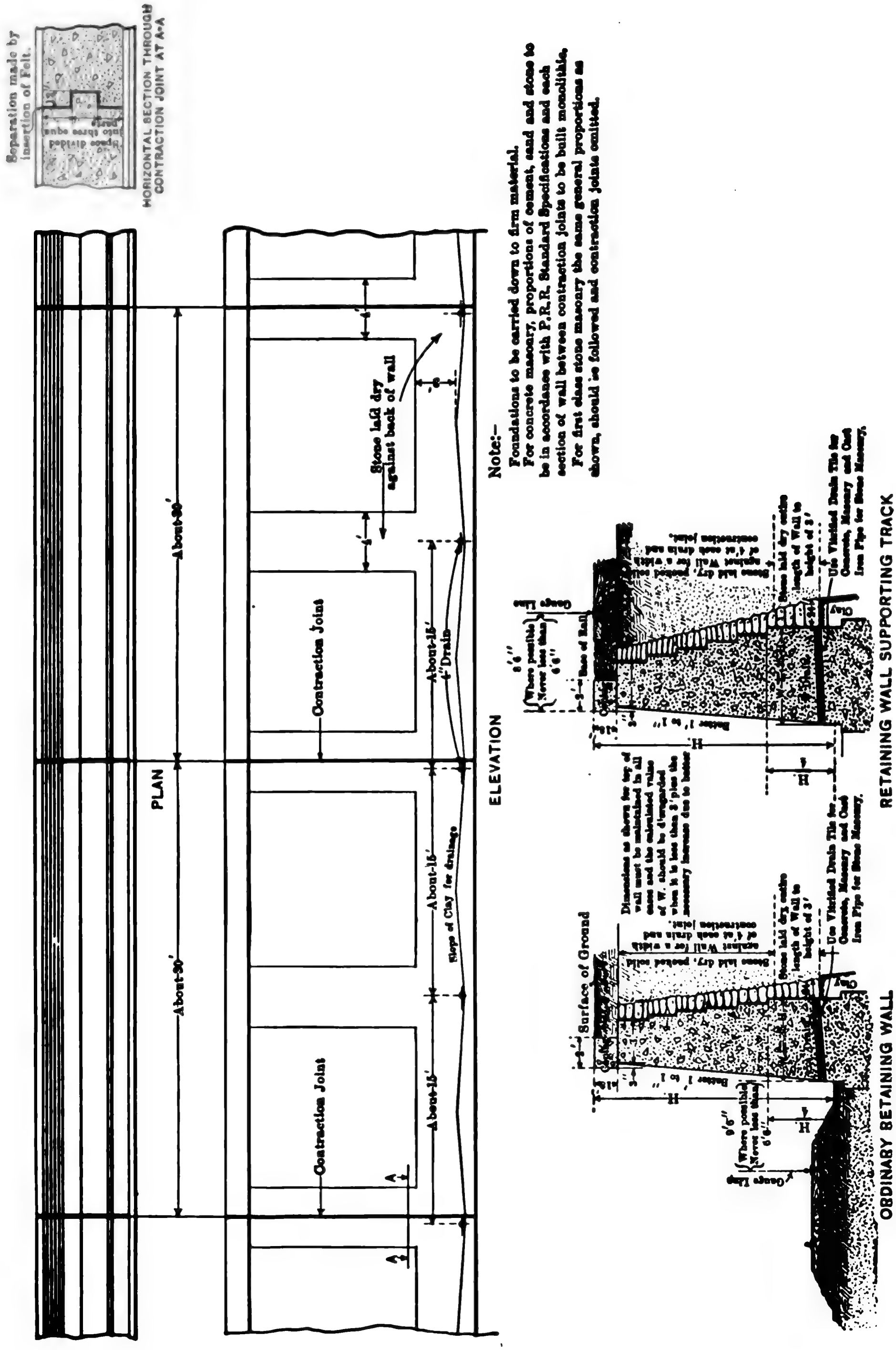


FIG. 221.—RETAINING WALLS, P. R. R. STANDARD.

but when of first-class stone, they are to be built continuous without expansion joints. Drainage is to be provided for by laying loose stone back of the wall as shown, and providing 4-inch drain pipes about every 15 feet as indicated.

The walls must be investigated by considering the concrete to weigh 150 pounds per cubic foot, the earth at 100 pounds per cubic foot, and the surcharge also at 100 pounds per cubic foot. Where the wall is an abutment or abutment pier then the formulas given in Chapter XVI shall be used.

Reinforced Concrete Highway Culverts

The standard reinforced concrete culverts adopted by the Los Angeles County Highway Commission are shown in Fig. 222, and

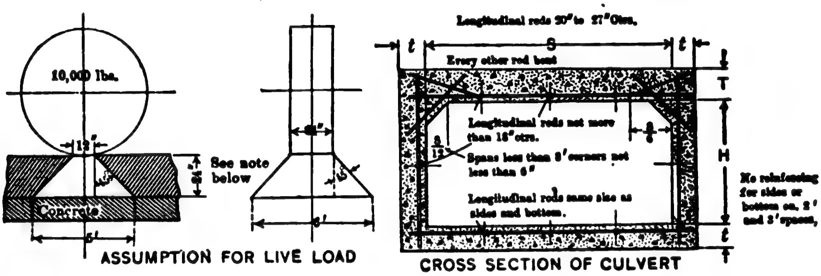


FIG. 222.—REINFORCED CONCRETE HIGHWAY CULVERTS.

the sizes shown in Table XXXV, were taken from an account in the *Engineering News*, by F. H. Joyner, Chief Engineer. Data are given as to size of rectangular culverts, reinforcement, quantities of steel, and yardage of concrete per lineal foot of culvert. The loading used is shown in Fig. 222, and consists of a 15-ton road roller, with 10,000 pounds on each wheel. The unit stresses used are 16,000 pounds per square inch for steel, 600 pounds for concrete, and 0.7 per cent of steel. The designs are made for a minimum fill of 8 inches of rock over culvert slabs, and are correct for all fills up to 6 feet in height. The designs represent first-class highway practice, and will be found useful for preliminary estimates and for preparing working plans.

Reinforced Concrete Railway Culverts

The standard reinforced concrete railway culverts for E50 and E60 loading as designed by the Grand Trunk Railway, are shown in Figs. 223 and 224, and the data in regard to the reinforcing, amount

TABLE XXXV.—REINFORCEMENT FOR STANDARD CULVERTS

d = depth of fill;
H = height of culvert;
T = thickness of top;
f = thickness of bottom and sides;
S = span.

				Top Reinforcement.			Corner Reinforcement.			Side Walls Reinforcement.			Bottom Reinforcement.			Quantities per Linear Foot of Box.		
d	H	T	f	Size.	Space.	Length.	Size.	Space.	Length.	Size.	Space.	Length.	Size.	Space.	Length.	Cu. Yds. Con.	Steel. Pounds.	
8″	1′	4″	4″	3/8″	8″	2′ 6″	....	....	....	....	....	....	....	....	....	.091	3.70	2′ span
8″	2′	4″	4″	3/8″	8″	2′ 6″	....	....	....	....	....	....	....	....	....	.115	3.70	
8″	1′	5½″	5″	3/8″	5″	3′ 9″	....	....	....	....	....	....	....	....	....	.155	7.17	3′ span
8″	2′	5½″	5″	3/8″	5″	3′ 9″	....	....	....	....	....	....	....	....	....	.186	7.17	
8″	3′	5½″	5″	3/8″	5″	3′ 9″	....	....	....	....	....	....	....	....	....	.216	8.12	
8″	1′	6″	5″	1/2″	8″	4′ 8″	3/8″	16″	1′ 6″	3/8″	16″	1′ 9″	3/8″	16″	4′ 8″	.204	12.83	4′ span
8″	2′	6″	5″	1/2″	8″	4′ 8″	3/8″	16″	1′ 6″	3/8″	16″	2′ 9″	3/8″	16″	4′ 8″	.235	13.55	
8″	3′	6″	5″	1/2″	8″	4′ 8″	3/8″	16″	1′ 6″	3/8″	16″	3′ 9″	3/8″	16″	4′ 8″	.2266	15.22	
8″	2′	6½″	5″	1/2″	7″	5′ 8″	3/8″	14″	1′ 6″	3/8″	14″	2′ 9″	3/8″	14″	5′ 8″	.278	17.91	5′ span
8″	3′	6½″	5″	1/2″	7″	5′ 8″	3/8″	14″	1′ 6″	3/8″	14″	3′ 9″	3/8″	14″	5′ 8″	.309	19.68	
8″	4′	6½″	5″	1/2″	7″	5′ 8″	3/8″	14″	1′ 6″	3/8″	14″	4′ 9″	3/8″	14″	5′ 8″	.339	20.49	
8″	2′	7½″	6″	1/2″	6″	6′ 10″	3/8″	12″	1′ 9″	3/8″	12″	3′ 0″	3/8″	12″	6′ 10″	.373	23.23	6′ span
8″	3′	7½″	6″	1/2″	6″	6′ 10″	3/8″	12″	1′ 9″	3/8″	12″	4′ 0″	3/8″	12″	6′ 10″	.410	25.14	
8″	4′	7½″	6″	1/2″	6″	6′ 10″	3/8″	12″	1′ 9″	3/8″	12″	5′ 0″	3/8″	12″	6′ 10″	.448	26.10	
8″	2′	8½″	6″	5/8″	8″	8′ 10″	1/2″	16″	2′ 0″	1/2″	16″	3′ 2″	1/2″	16″	8′ 10″	.494	38.55	8′ span
8″	3′	8½″	6″	5/8″	8″	8′ 10″	1/2″	16″	2′ 0″	1/2″	16″	4′ 2″	1/2″	16″	8′ 10″	.531	41.30	
8″	4′	8½″	7″	5/8″	8″	9′ 0″	1/2″	16″	2′ 0″	1/2″	16″	5′ 2″	1/2″	16″	9′ 0″	.628	43.02	
8″	5′	8½″	7″	5/8″	8″	9′ 0″	1/2″	16″	2′ 0″	1/2″	16″	6′ 2″	1/2″	16″	9′ 0″	.671	46.00	
8′	2′	10″	7″	5/8″	6½″	11′ 0″	1/2″	13″	2′ 6″	1/2″	13″	3′ 3″	1/2″	13″	11′ 0″	.700	54.90	10′ span
8″	3′	10″	7″	5/8″	6½″	11′ 0″	1/2″	13″	2′ 6″	1/2″	13″	4′ 3″	1/2″	13″	11′ 0″	.742	58.17	
8″	4′	10″	7″	5/8″	6½″	11′ 0″	1/2″	13″	2′ 6″	1/2″	13″	5′ 3″	1/2″	13″	11′ 0″	.786	59.74	
8″	5′	10″	8″	5/8″	6½″	11′ 2″	1/2″	13″	2′ 8″	1/2″	13″	6′ 4″	1/2″	13″	11′ 2″	.900	63.90	
8″	6′	10″	8″	5/8″	6½″	11′ 2″	1/2″	13″	2′ 8″	1/2″	13″	7′ 4″	1/2″	13″	11′ 2″	.949	65.47	
8″	7′	10″	8″	5/8″	6½″	11′ 2″	1/2″	13″	2′ 8″	1/2″	13″	8′ 4″	1/2″	13″	11′ 2″	1.000	68.74	
8″	8′	10″	8″	5/8″	6½″	11′ 2″	1/2″	13″	2′ 8″	1/2″	13″	9′ 4″	1/2″	13″	11′ 2″	1.048	70.31	
8″	2′	11″	8″	3/4″	6″	13′ 2″	1/2″	12″	3′ 0″	1/2″	12″	3′ 7″	1/2″	12″	13′ 2″	.916	69.31	12′ span
8″	3′	11″	8″	3/4″	6″	13′ 2″	1/2″	12″	3′ 0″	1/2″	12″	4′ 7″	1/2″	12″	13′ 2″	.965	72.71	
8″	4′	11″	8″	3/4″	6″	13′ 2″	1/2″	12″	3′ 0″	1/2″	12″	5′ 7″	1/2″	12″	13′ 2″	1.014	74.41	
8″	5′	11″	8″	3/4″	6″	13′ 2″	1/2″	12″	3′ 0″	1/2″	12″	6′ 7″	1/2″	12″	13′ 2″	1.063	77.81	
8″	6′	11″	9″	3/4″	6″	13′ 4″	1/2″	12″	3′ 0″	1/2″	12″	7′ 6″	1/2″	12″	13′ 4″	1.205	79.94	
8″	7′	11″	9″	3/4″	6″	13′ 4″	1/2″	12″	3′ 0″	1/2″	12″	8′ 6″	1/2″	12″	13′ 4″	1.261	83.34	
8″	8′	11″	9″	3/4″	6″	13′ 4″	1/2″	12″	3′ 0″	1/2″	12″	9′ 6″	1/2″	12″	13′ 4″	1.316	85.04	
8″	9′	11″	9″	3/4″	6″	13′ 4″	1/2″	12″	3′ 0″	1/2″	12″	10′ 6″	1/2″	12″	13′ 4″	1.372	88.44	
8″	10′	11″	9″	3/4″	6″	13′ 4″	1/2″	12″	3′ 0″	1/2″	12″	11′ 6″	1/2″	12″	13′ 4″	1.427	90.14	

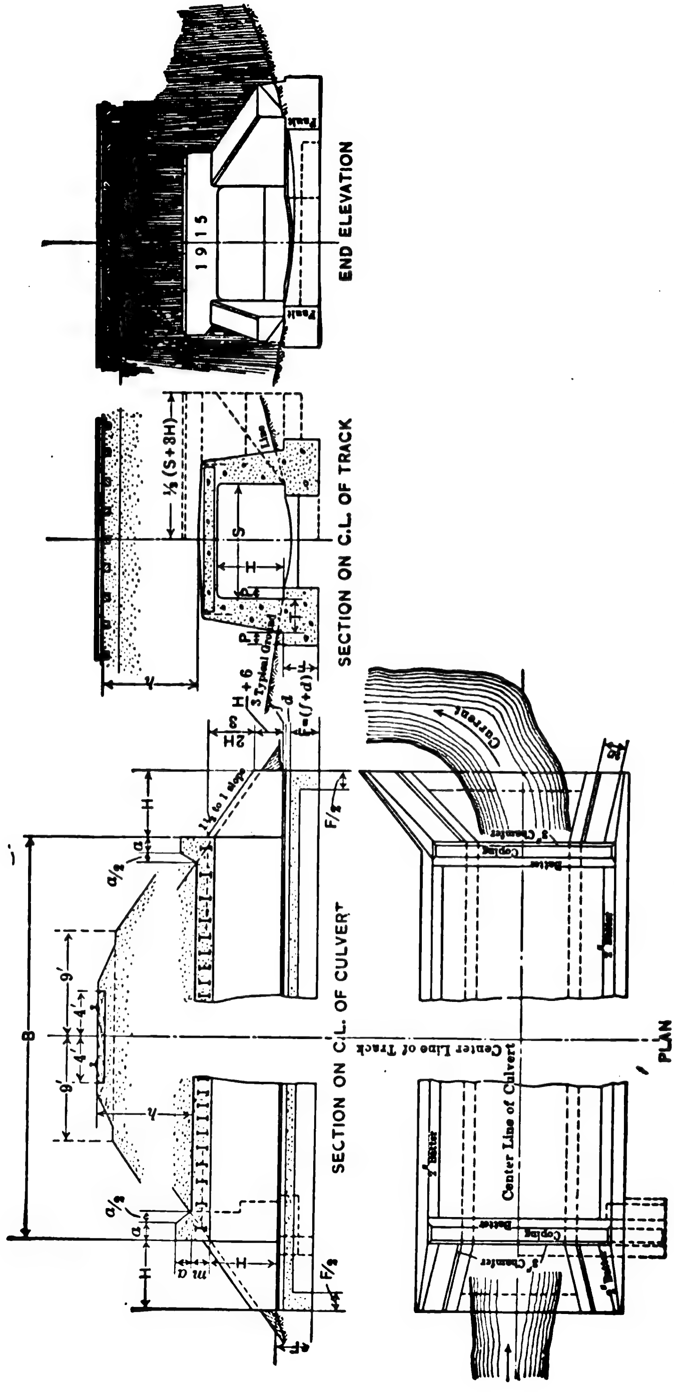


FIG. 223.—STANDARD BEAM TOP CULVERT, GRAND TRUNK RY.

of steel and concrete, and all of the dimensions are given in Tables XXXVI to XLI.

The abutments are to be built first up to the dotted line AA only, the steel dowels being set as shown. The beams are to be then placed, and all of the remaining concrete is to be cast monolithic. The 2-inch protection under the beams is to be of fine concrete or mortar. The top of the cover is to be finished smooth with fine concrete, and waterproofed with 5-ply 10-ounce felt and coal-tar pitch. In case of culverts having only 1 foot 6 inches of ballast over the cover, this waterproofing is to be protected by 2 inches of cement mortar throughout. The waterproofing is to be carried sufficiently far down the abutment to thoroughly cover the joint AA; and at the

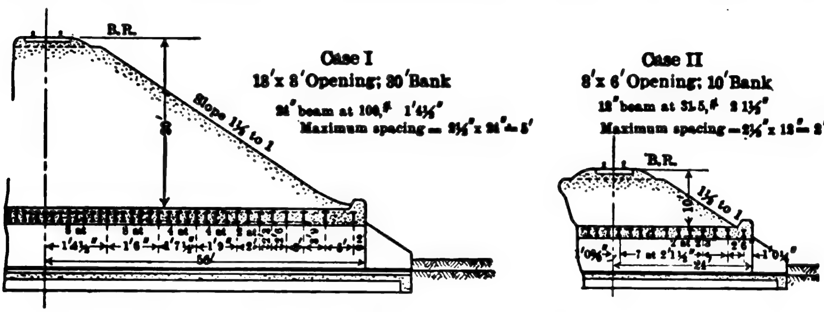


FIG. 224.—TYPICAL BEAM SPACING, GRAND TRUNK RY. CULVERTS.

ends of the barrel, it is to be carried up to the parapet walls above the slope of the fill, and finished in a groove provided for it.

Joint locks of suitable size, are to be molded in the footing; and stone dowels are to be set in each lift of the abutment walls, or at whatever point the work leaves off at night. In cases where the stream takes a sharp turn, either at the entry to the culvert or on leaving it, or both, one or more wings are to be splayed as required; the angle and length being varied to best suit the conditions. The angle is usually 30 or 45 degrees. The concrete dish is to be given a fall towards the downstream end, f, equal to about $\frac{1}{16}$ inch per foot, but not less than 1½ inches, nor more than 6 inches in the total length of the barrel, except in special cases of rapid fall.

The footing dimensions given are the minimum necessary to give foundations loads not to exceed 2½ tons per square foot, and in cases of hard material or rock they may be reduced in size. Any increase necessary in a particular case to provide the required bearing area will be evened up to the nearest 3 inches and 1½ inches for depth

TABLE XXXVI.—GRAND TRUNK RAILWAY CULVERTS

GENERAL DIMENSIONS

	Size of Opening. Span by Height.						
	5×3	5×4	5×5	6×4	6×6	8×4	8×8
Area of opening, square feet	15	20	25	24	36	32	64
Length of beam bearing (*n*)	12″	12″	12″	12″	12″	1′ 1½″	1′ 1½″
Height and width of parapet (*a*)	1′ 3″	1′ 3″	1′ 3″	1′ 3″	1′ 6″	1′ 6″	1′ 6″
Thickness of paving (*l*)	6″	6″	6″	6″	6″	7½″	7½″
Depth of dish (*d*)	1½″	1½″	1½″	1½″	1½″	3″	3″
Crown of cover (*c*)	3″	3″	3″	3″	3″	3″	3″
Size of steel dowels	¾″ ○	¾″ ○	¾″ ○	¾″ ○	¾″ ○	¾″ □	¾″ □
Length of steel dowels	1′ 3″	1′ 3″	1′ 3″	1′ 6″	1′ 6″	2′ 0″	2′ 0″
Width of wing coping (*w*)	1′ 3″	1′ 3″	1′ 3″	1′ 3″	1′ 3″	1′ 6″	1′ 6″
Depth of footing (*F*)	2′ 6″	2′ 6″	2′ 6″	2′ 6″	2′ 6″	2′ 9″	2′ 9″
Footing projection (*p*)	6″	6″	6″	7½″	7½″	9″	9″

	10×4	10×8	12×6	12×10	14×6	14×10	15×6
Area of opening, square feet	40	80	72	120	84	140	90
Length of beam bearing (*n*)	1′ 3″	1′ 3″	1′ 4½″	1′ 4½″	1′ 6″	1′ 6″	1′ 7½″
Height and width of parapet (*a*)	1′ 6″	1′ 6″	1′ 9″	1′ 9″	1′ 9″	1′ 9″	1′ 9″
Thickness of paving (*l*)	7½″	7½″	9″	9″	9″	9″	10½″
Depth of dish (*d*)	3″	3″	4½″	4½″	4½″	4½″	4½″
Crown of cover (*c*)	3″	3″	3″	3″	4½″	4½″	4½″
Size of steel dowels	¾″ □	¾″ □	1″ ○	1″ ○	1″ ○	1″ ○	1″ ○
Length of steel dowels	2′ 6″	2′ 6″	2′ 6″	2′ 6″	3′ 0″	3′ 0″	3′ 0″
Width of wing coping (*w*)	1′ 6″	1′ 6″	1′ 9″	1′ 9″	1′ 9″	1′ 9″	1′ 9″
Depth of footing (*F*)	2′ 9″	2′ 9″	3′ 0″	3′ 0″	3′ 3″	3′ 3″	3′ 3″
Footing projection (*p*)	10½″	10½″	1′ 0″	1′ 0″	1′ 1½″	1′ 1½″	1′ 1½″

	15×10	16×6	16×10	18×6	18×10	20×6	20×10
Area of opening, square feet	150	96	160	108	180	120	200
Length of beam bearing (*n*)	1′ 7½″	1′ 9″	1′ 9″	1′ 10½″	1′ 10½″	2′ 0″	2′ 0″
Height and width of parapet (*a*)	1′ 9″	2′ 0″	2′ 0″	2′ 0″	2′ 0″	2′ 0″	2′ 0″
Thickness of paving (*l*)	10½″	10½″	10½″	12″	12″	12″	12″
Depth of dish (*d*)	4½″	6″	6″	6″	6″	6″	6″
Crown of cover (*c*)	4½″	4½″	4½″	6″	6″	6″	6″
Size of steel dowels	1″ ○	1″ □	1″ □	1″ □	1″ □	1″ □	1″ □
Length of steel dowels	3′ 0″	3′ 6″	3′ 6″	3′ 6″	3′ 6″	4′ 0″	4′ 0″
Width of wing coping (*w*)	1′ 9″	2′ 0″	2′ 0″	2′ 0″	2′ 0″	2′ 0″	2′ 0″
Depth of footing (*F*)	3′ 3″	3′ 6″	3′ 6″	3′ 9″	3′ 9″	4′ 0″	4′ 0″
Footing projection (*p*)	1′ 1½″	2@7½″	2@7½″	2@9″	2@9″	2@10½″	2@10½″

and projection respectively. The quantities given in the tables are based on these dimensions. The dimensions for the footings must be considered independently for each case after test pits have been dug or borings made, and the depth and spread of the footing determined according to standard bearing values. Piles are to be figured by the *Engineering News* formula for direct load; but if they act

TABLE XXXVII.—GRAND TRUNK RAILWAY CULVERTS

Details of Beams. E60 Live Load

Clear Span.	Height of Bank. (B. R. to Top of Crown of Cover.)															
	1' 6"				5' 0"				10' 0"				15' 0"			
'	"	#	'	"	"	#	'	"	"	#	'	"	"	#	'	"
5	8	18	1	1½	8	18	1	4½	8	18	1	6	8	18	1	6
6	9	21	1	3	9	21	1	6	9	21	1	7½	9	21	1	7½
8	12	31½	1	9	12	31½	2	1½	12	31½	2	1½	12	31½	1	10½
10	15	42	2	1½	15	42	2	3	15	42	2	3	15	42	2	0
12	15	42	1	4½	15	42	1	7½	15	42	1	7½	15	42	1	4½
14	18	55	1	7½	18	55	1	9	18	55	1	9	18	55	1	7½
15	20	65	2	0	20	65	2	1½	20	65	2	0	20	65	1	10½
16	20	65	1	9	20	65	1	10½	20	65	1	9	20	65	1	7½
18	20	65	1	4½	20	65	1	6	20	65	1	4½	20	65	1	3
20	24	80	1	9	24	80	1	10½	24	80	1	9	24	80	1	6

	20' 0"				30' 0"				40' 0"				50' 0"			
'	"	#	'	"	"	#	'	"	"	#	'	"	"	#	'	"
5	8	18	1	4½	9	21	1	6	10	25	1	6	10	25	1	3
6	10	25	1	9	12	31½	2	1½	12	31½	1	9	15	42	2	3
8	12	31½	1	7½	15	42	2	0	15	42	1	7½	18	55	2	0
10	15	42	1	9	15	42	1	4½	18	55	1	7½	18	55	1	4½
12	18	55	1	10½	18	55	1	4½	20	65	1	6	20	65	1	3
14	20	65	1	9	20	65	1	4½	24	80	1	7½	24	80	1	4½
15	20	65	1	6	24	80	1	9	24	80	1	4½	24	80	1	1½
16	20	65	1	4½	24	80	1	6	24	80	1	3	24	100	1	1½
18	24	80	1	7½	24	100	1	4½	24	100	1	1½	24	105	1	1½
20	24	80	1	3	24	100	1	1½	24	105	1	1½	24	115	1	0

Note.—The figures given above indicate, in each case, the depth of beam, its weight per foot length, and the spacing center to center, respectively. For example, a culvert of 12' span, having 5' 0" of bank between the top of the cover and B. R., will require a beam, 15" deep, weighing 42 lbs. per lin. ft., and spaced 1' 7½" center to center. The spacings given are the maximum allowable at the center of the culvert, where the height of bank is greatest. They may be increased towards the ends of the culvert, but the maximum spacing must not exceed 2½ times the depth of the beam. The beams given above are Cambria sections, but any beams having an equivalent section modulus may be used.

as columns, then the total length is to be assumed as reduced from 25 to 33 per cent on account of the support afforded by the surrounding material.

The design of the beams is made on the assumption that the live load spreads naturally from the ends of an 8-foot tie at a ½ to 1 slope through the fill to the concrete, and from there at 1 to 1 slopes through the concrete to the middle height of the beams. The middle height is used since the beam carries the load partly on its upper and partly on its lower flanges. The formation of a natural arch in the new fill over the culvert was not considered sufficiently probable to

TABLE XXXVIII.—GRAND TRUNK RAILWAY CULVERTS

DETAILS OF BEAMS. E50 LIVE LOAD

Clear Span.	Height of Bank. (B. R. to Top of Crown of Cover.)															
	1′ 6″				5′ 0″				10′ 0″				15′ 0″			
′	″	#	′	″	″	#	′	″	″	#	′	″	″	#	′	″
5	8	18	1	4½	8	18	1	7½	8	18	1	9	8	18	1	7½
6	9	21	1	6	9	21	1	9	9	21	1	10½	9	21	1	7½
8	10	25	1	4½	10	25	1	7½	10	25	1	6	10	25	1	4½
10	12	31.5	1	6	12	31.5	1	7½	12	31.5	1	6	12	31.5	1	4½
12	15	42	1	7½	15	42	1	10½	15	42	1	9	15	42	1	6
14	18	55	2	0	18	55	2	1½	18	55	1	10½	18	55	1	7½
15	18	55	1	9	18	55	1	10½	18	55	1	7½	18	55	1	4½
16	18	55	1	6	18	55	1	7½	18	55	1	6	20	65	1	9
18	20	65	1	7½	20	65	1	9	20	65	1	6	20	65	1	4½
20	20	65	1	4½	24	80	2	1½	24	80	1	10½	24	80	1	7½

NOTE.—For heights of bank greater than 15 feet, use the figures as given for E60, in table A, the difference in the two being negligible beyond this point. For explanation of the figures see sheet 4.

All tables have been prepared on the assumption that the bank is composed of average material having a natural slope of 1½ to 1. Any special case of a culvert situated in a bank made up of material varying considerably from this must receive careful consideration.

materially affect the design of a standard structure for general use, although it might occur in high banks after years of consolidation and might therefore be taken into account in special cases.

The beams are considered as simply supported at the ends, and the same size beam has been used at the ends with a wider spacing rather than to use smaller beams. The thickness of the wall has been determined by considering it as an eccentrically loaded column, adding 50 per cent, and giving it a 2-inch batter at the back to spread the load more uniformly over the base. The loads used were E50 and E60 for live load; and for dead loads, concrete 150 pounds per cubic foot, earth fill 100 pounds per cubic foot, and the steel at the actual weight. Impact was taken account of by the formula $L^2 \div L+D$.

The above is taken from the standard tables of the Grand Trunk Railway, prepared under the direction of F. L. C. Bond, Chief Engineer, by R. Armour, Masonry Engineer.

Flat Reinforced Concrete Slabs

Practical rules for applying results of tests of full size reinforced concrete slabs in the design of bridges are given by Mr. A. T. Goldbeck,

TABLE XXXIX.—GRAND TRUNK RAILWAY CULVERTS

THICKNESS OF ABUTMENT (T)

Height of Bank. (B. R. to Top of Crown of Cover.)	Size of Opening. Span by Height.						
	5×3	5×4	5×5	6×4	6×6	8×4	8×8
Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.
1 6	2 1½	2 3	2 6	2 6	2 9	2 7½	3 3
5 0	2 0	2 1½	2 3	2 3	2 6	2 4½	3 0
10 0	1 10½	2 0	2 3	2 3	2 6	2 4½	3 0
15 0	1 10½	2 0	2 3	2 3	2 6	2 4½	3 0
20 0	2 0	2 1½	2 3	2 3	2 6	2 6	3 1½
30 0	2 1½	2 3	2 6	2 6	2 9	2 9	3 4½
40 0	2 4½	2 6	2 9	2 9	3 1½	3 1½	3 9
50 0	2 7½	2 9	3 0	3 0	3 4½	3 4½	4 1½

	10×4	10×8	12×6	12×10	14×6	14×10	15×6
Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.
1 6	2 9	3 6	3 3	4 0	3 4½	4 0	3 4½
5 0	2 6	3 1½	3 0	3 9	3 1½	3 9	3 3
10 0	2 6	3 1½	3 0	3 9	3 1½	3 9	3 3
15 0	2 7½	3 3	3 1½	3 10½	3 3	4 0	3 4½
20 0	2 10½	3 6	3 4½	4 0	3 6	4 1½	3 7½
30 0	3 3	3 10½	3 9	4 4½	4 0	4 7½	4 1½
40 0	3 6	4 1½	4 1½	4 9	4 6	5 1½	4 7½
50 0	3 10½	4 6	4 7½	5 3	5 0	5 7½	5 3

	15×10	16×6	16×10	18×6	18×10	20×6	20×10
Ft. In.	Ft. In.	Ft. In.	Ft. In.	Fr. In.	Ft. In.	Ft. In.	Ft. In.
1 6	4 1½	3 6	4 1½	3 7½	4 3	3 9	4 4½
5 0	[illegible] 10½	3 4½	4 0	3 6	4 1½	3 7½	4 3
10 0	3 10½	3 4½	4 0	3 7½	4 3	3 9	4 4½
15 0	4 1½	3 6	4 1½	3 9	4 6	4 0	4 7½
20 0	4 4½	3 9	4 6	4 0	4 9	4 4½	5 3
30 0	4 10½	4 3	5 0	4 7½	5 3	5 0	5 9
40 0	5 4½	4 10½	5 6	5 3	5 10½	5 7½	6 3
50 0	5 10½	5 4½	6 1½	5 9	6 6	6 1½	6 10½

NOTE.—All tables have been prepared on the assumption that the bank is composed of average material having a natural slope of 1½ to 1. Any special case of a culvert situated in a bank made up of material varying considerably from this, must receive careful consideration.

Engineer of Tests, U. S. Bureau of Public Roads, in the September, 1918, issue of *Public Roads*.

In all of the slab tests at the Bureau of Public Roads the method of procedure was to apply known concentrated loads on the slab specimens which rested on two supports. The deformation of the steel reinforcing and concrete, and also the deflection were measured. These deformations or changes in length in the slab were always

TABLE XL.—GRAND TRUNK RAILWAY CULVERTS

CONCRETE QUANTITIES

Height of Bank.		5×3	5×4	5×5	6×4	6×6	8×4	8×8	10×4	10×8	12×6	12×10
1′ 6″	*a*	1.40	1.57	1.83	1.84	2.24	2.30	3.28	2.71	3.83	3.72	5.02
	b	8	11.5	15	12.5	20	16.5	37.5	18.5	41	35.5	70
	c	38	46	55	52	68	68	111	81	129	121	185
5′ 0″	*a*	1.40	1.50	1.70	1.70	2.06	2.15	3.05	2.55	3.48	3.52	4.74
	b	8	11.5	15	12	18	15	36.5	18	39	34.5	69
	c	51	60	68	66	84	86	137	103	156	152	227
10′	*a*	1.28	1.43	1.67	1.70	2.06	2.15	3.05	2.55	3.48	3.52	4.74
	b	8	11.5	15	12	18	15	37	18	39	35	69
	c	68	79	93	92	115	119	183	142	208	205	299
15′	*a*	1.28	1.43	1.67	1.70	2.06	2.15	3.05	2.63	3.61	3.62	4.88
	b	8	11.5	15	12	18	15	37	18	40	35	69
	c	87	100	118	117	146	151	229	185	269	265	379
20′	*a*	1.34	1.50	1.67	1.73	2.09	2.24	3.17	2.78	3.83	3.97	5.17
	b	8	11.5	15	12	18	16	37	19	41	36	70
	c	113	127	144	146	179	190	285	237	342	351	480
30′	*a*	1.42	1.60	1.85	1.93	2.32	2.49	3.52	3.02	4.23	4.27	5.58
	b	8	11.5	15	12.5	20	16.5	38	20	43	37	71
	c	160	182	215	221	270	286	420	347	502	503	683
40′	*a*	1.58	1.78	2.03	2.06	2.59	2.71	3.82	3.27	4.61	4.68	6.12
	b	8.5	12	16	13	21	18	40	21	46	38	73
	c	226	256	295	297	379	392	569	476	679	693	930
50′	*a*	1.70	1.91	2.20	2.32	2.90	3.00	4.31	3.45	4.90	5.08	6.66
	b	9	12	16.5	13.5	24	19	41	22	47	40	75
	c	294	332	385	404	509	528	771	607	878	904	1207

Height of Bank.		14×6	14×10	15×6	15×10	16×6	16×10	18×6	18×10	20×6	20×10
1′ 6″	*a*	4.37	5.58	4.65	6.00	4.96	6.23	5.63	6.93	6.49	7.84
	b	40	75	42	79	47	86	54	96	60	104
	c	147	212	158	229	171	242	194	269	229	307
5′ 0″	*a*	4.16	5.30	4.54	5.72	4.86	6.08	5.51	6.78	6.37	7.69
	b	39	73	41	77	47	85	53	95	60	103
	c	184	259	203	280	219	301	249	336	292	384
10′	*a*	4.16	5.30	4.54	5.72	4.86	6.08	5.63	6.93	6.49	7.84
	b	39	73	41	77	47	85	54	96	60	103
	c	217	338	271	366	292	392	338	446	394	508
15′	*a*	4.26	5.58	4.65	6.00	4.96	6.23	5.73	7.23	6.65	8.22
	b	39	78	42	79	47	86	54	97	61	105
	c	316	437	346	472	372	494	430	570	503	652
20′	*a*	4.58	5.90	4.86	6.28	5.17	6.65	6.25	7.81	7.07	8.85
	b	40	76	43	81	48	88	55	98	63	109
	c	409	578	434	586	467	624	565	735	639	830
30′	*a*	5.00	6.31	5.54	7.11	5.88	7.50	6.82	8.41	7.65	9.53
	b	43	79	45	84	50	91	58	101	66	112
	c	595	776	663	877	705	927	818	1041	918	1174
40′	*a*	5.68	7.25	6.17	7.70	6.44	8.10	7.38	9.17	8.23	10.14
	b	45	83	48	88	52	93	61	104	69	115
	c	849	1108	921	1178	963	1239	1105	1402	1233	1550
51′	*a*	6.11	7.80	6.51	8.29	6.88	8.84	7.84	9.92	8.69	10.9
	b	47	86	51	92	54	96	63	107	71	118
	c	1095	1425	1168	1513	1235	1612	1407	1809	1561	1987

NOTE.—The line (*a*) gives the volume of concrete in the barrel per foot run; (*b*) gives the total volume of the portions outside the ends of the barrel; and (*c*) gives the volume of the complete culvert. All quantities given are in cubic yards.

TABLE XLI.—GRAND TRUNK RY. CULVERTS

TOTAL WEIGHT OF STEEL

(Including Dowels, Bolts etc.)

Height of Bank (B. R. to top of cover).	Clear Span.									
	5′	6′	8′	10′	12′	14′	15′	16′	18′	20′
1′ 6″ [Ballast only]	2,500	2,950	4,350	5,900	10,150	14,300	14,450	18,100	24,550	27,250
5′ 0″	2,900	3,700	5,500	8,000	13,500	19,000	21,000	23,250	32,500	37,000
10′ 0″	3,950	4,850	7,250	10,200	17,150	23,850	26,500	31,000	40,000	46,750
15′ 0″	5,100	6,600	10,300	14,500	23,450	32,000	36,500	41,500	56,100	61,900
20′ 0″	6,650	8,450	16,950	20,900	29,000	41,500	50,500	60,750	68,850	95,400
30′ 0″	9,550	11,950	21,500	31,000	50,350	61,800	71,600	87,500	129,200	177,500
40′ 0″	14,200	17,000	30,300	47,900	73,000	94,700	110,000	120,000	165,000	228,000
50′ 0″	18,800	22,100	35,500	62,000	101,500	119,000	143,000	181,000	216,000	286,000

NOTE.—The quantities given on this page, and also the concrete quantities given on page 358, are based on the assumption that the natural slope of the bank is 1½ to 1. They are for single-track embankments.

taken at the "dangerous section," where they were greatest. In a few cases, deformations were also measured over the entire area of the slab. A strain gage capable of measuring changes of 0.0002 of an inch was used in all of the tests and in addition, the vertical deflections of the slab were obtained, generally by means of a micrometer head reading to 0.001 of an inch.

Consider first a wide slab supporting a single load concentrated at its center. The maximum deformation occurs under the load, and as the sides of the slab are approached the deformation becomes smaller. This curve of deformation is the same in shape for both the steel and concrete. The resisting moment of any slab is directly proportional to the area of the curve of unit deformation. Two similar slabs stressed to have the same area of unit deformation, even though their unit deformation curves are dissimilar in shape, exert equal resisting moments. The effective width of the slab is the width which may be considered as carrying the entire concentrated load. When the value for this width as determined by test is substituted in the common formulas for narrow rectangular beams, these formulas may be directly applied to the design of wide slabs.

A number of slabs have been tested as outlined above, and their effective widths have been obtained from the deformation curves, first, by getting the areas included between these curves and their base lines, then dividing these areas by their maximum ordinates. When the load is placed in the centers of the slab and the width of the slab is more than about twice the span length, the effective width may be considered as equal to seven-tenths of the span length of the slab.

A number of slabs have been tested with a central load and having width equal to twice their span lengths. The table below gives data on the slabs tested at the Bureau of Public Roads during the past five years.

DATA ON SLAB TESTS OF BUREAU OF PUBLIC ROADS

Slab No.	Dimensions.		Depth.		Steel Percentage.		Central Load, Effective Width ÷ Span.	Failure.	
	Span, Feet.	Breadth, Feet.	Total, Inches.	Effective, Inches.	Longitudinal.	Transverse.		Span.	Central Load.
679	11.5	6	7	6	0.77	....	......	11.5	21,500
705	6	7	5	4	.91	....	0.9	....	......
706	3	7	5	4	.91	0.41	......	3	42,800
730	5	7	6	5	.91	....	1.1	6	24,700
	6	....	....	....	....	....	.96	....	......
736	6	7	4	3	.60	....	......	6	7,560
737	5	7	7	6	.75	.33	1.2	6	34,200
	6	....	....	....	....	....	1.2	....	......
835	16	32	12	10¼	.75	....	*	16	119,000
930	16	32	10	8¼	.75	....	*	16	80,000
934	16	32	7	6	.75	....	*	16	40,000

* See next table.

EFFECTIVE WIDTHS UNDER CENTRAL LOADS

Center Load.	Slab 835: 10¼ Inches Effective Depth.	Slab 930: 8¼ Inches Effective Depth.	Slab 934: 6 Inches Effective Depth.
15,000	....	11′.4=71.6% span	12′.7=79.5% span
20,000	11′.6=72.3% span	13′.0=81.2% span	17′.5=109.3% span
25,000	11′.5=71.9% span	12′.9=81 1% span	
32,500	12′.1=75.7% span		
35,000	....	14′.5=90.7% span	
Failure	119,000 lbs.	80,000 lbs.	40,000 lbs.

The foregoing discussion treats of slabs having widths equal to twice the span length, in which case the sides of the slabs are not stressed appreciably. When the width is less than this, however, stress does reach the sides, and the narrower the slab, the more are the sides put under stress. It will be recognized that the width of the slab plays an important part in influencing the effective width. The amount of this influence has been quite fully investigated by a number of slab tests, in which the width of the specimen has been decreased after each load application, the sides of the slab having been split off by means of plugs and feathers. It has been possible

to obtain from these investigations the values for effective width given in the following list. These values also are plotted in Fig. 225.

VALUES FOR EFFECTIVE WIDTH

Total Width ÷ Span.	Effective Width ÷ Span.	Total Width ÷ Span.	Effective Width ÷ Span.
0.1	0.1	1.1	0.67
0.2	0.2	1.2	0.68
0.3	0.28	1.3	0.70
0.4	0.37	1.4	0.71
0.5	0.44	1.5	0.72
0.6	0.50	1.6	0.72
0.7	0.55	1.7	0.72
0.8	0.58	1.8	0.72
0.9	0.62	1.9	0.72
1.0	0.65	2.0	0.72

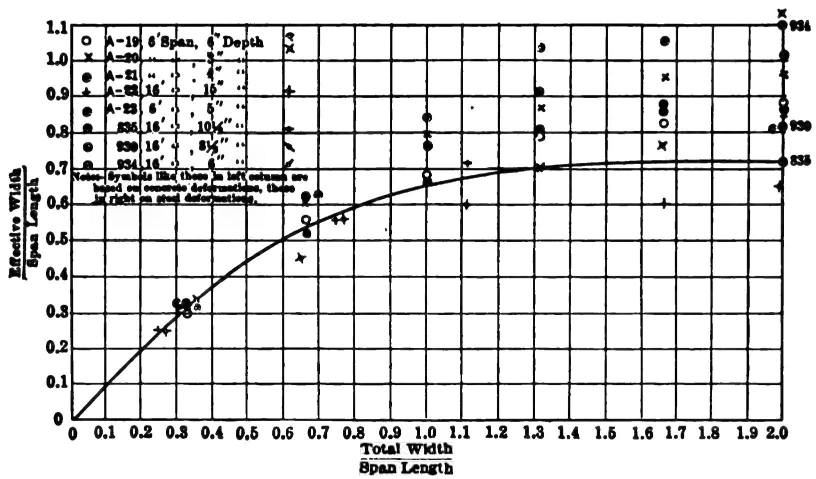


FIG. 225.—INFLUENCE OF TOTAL WIDTH ON EFFECTIVE WIDTH OF REINFORCED CONCRETE SLABS WITH CONCENTRATED LOADING.

The above values may be used for spans up to 16 feet at least, and probably for longer spans.

When the span is such that a single axle load will control the maximum bending moment, the slab is subjected to two wheel concentrations, and the most dangerous condition exists when these wheels are midway between the supports. In order to investigate this condition, tests were made on slabs with two loads spaced 5 feet apart on the center line of the slab.

Note the fact that directly under the load the deformations are greatest, and are even slightly greater than the deformation at the

center of the slab. This stress distribution does not hold, however, for every thickness of slab, for a few of the tests show the deformation to be greatest at the center. The effective width of slabs loaded in this way may, in general, be assumed as equal to the effective width due to a single load plus 4 feet.

When a heavy load traverses a slab bridge it may not remain at the center line, but may travel over the bridge near one side. Again there are often occasions where a heavy traction engine will

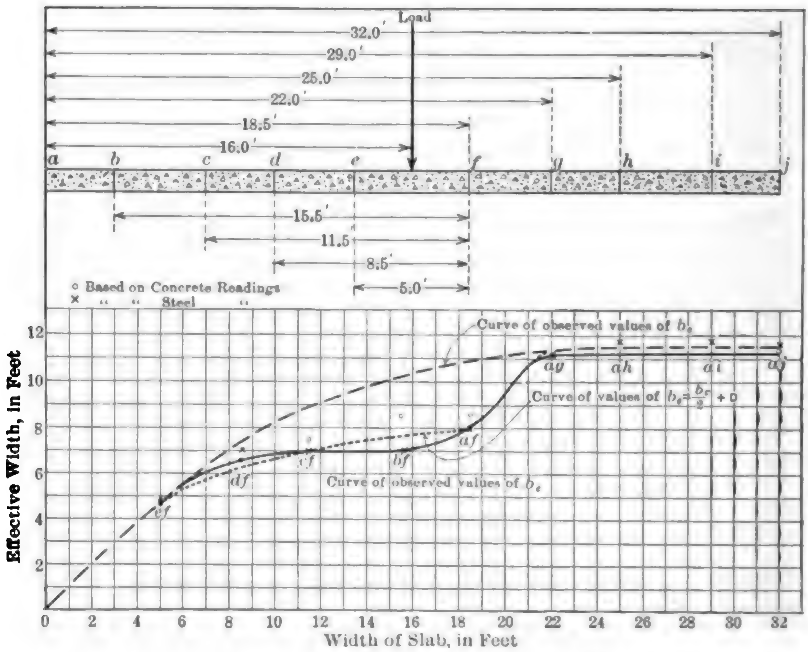


FIG. 225a.—CURVE SHOWING EFFECTIVE WIDTH VS. WIDTH OF SLAB.

stop at the side of a bridge spanning a stream, in order to replenish its supply of water. On such occasions heavy load concentrations are supported on the side of the bridge as eccentric loads, and this is a much more severe condition than that of the centrally applied load.

For the investigation of this case a slab specimen 16 feet in span, 32 feet in width and 13 inches in effective depth was made up. The original width was decreased after each test by cutting strips off of one side. The load applied in the center of the original 32-foot width became an eccentric load as the side of the slab was gradually removed. Referring to Fig. 225a, the strips *ij*, *hi*, *gh*, *fg*, *ab*, *bc*, *cd*, and *de*

were split off in the order named. The load was always applied to the same spot and thus its eccentricity raised as the slab width was decreased. Without going into the details of the test, the results are shown in the curve in Fig. 225*a*. The dash line curve is merely a duplication of that in Fig. 225, and applies to wide slabs under a central load. The solid curve is based on the tests of the eccentrically loaded slab. About 10 feet were split off the sides of the slab before the effective width began to differ from that of the centrally loaded slab. The load was then 6 feet from the side of the slab, and when its distance to the side became less than this, the effective width became much less than that of the same slab centrally loaded. This is shown by the deviation of the solid line from the dash line. The dotted line is plotted to represent the effective width of an eccentrically loaded slab with values for this effective width assumed to be equal to $\frac{b_c}{2}+D$, where b_c = the effective width of the slab under central load. D = distance of load to nearest side of slab.

This curve follows the curve of test results very closely and it may be quite safely stated as a general proposition that when a slab is eccentrically loaded, the effective width to be used in design may be calculated in the following manner:

(1) When the distance of the load from the nearest side is more than half of the effective width of the centrally loaded slab (see list) use the effective width for central loads.

(2) When the distance of the load from the side of the slab is less than half the effective width under central loads (see list) the effective width is to be taken equal to $\frac{b_c}{2}+D$. In order to make a slab bridge eccentrically loaded equal in strength to one centrally loaded, it will be necessary to supply extra strength at the sides by means of a parapet wall, and the following procedure for the design will give safe results:

(1) Use the formulas for narrow rectangular beams substituting for the breadth b the value obtained from the list; (2) determine the loss in effective width due to the assumed eccentricity of the load; (3) supply the deficiency by designing the curb of the parapet to provide a resisting moment equal to that of the slab width lost due to eccentricity. Allowance will have to be made, however, for the stiffness of the section under the parapet. An unfinished test thus far indicates that this method of design is safe at least.

To illustrate the above method. Assume the slab 16 feet in span length and 20 feet in width, designed to carry a concentrated load to be applied 3 feet from one edge, then the

$$\frac{\text{Total width}}{\text{span}}=\frac{20}{16}=1.25.$$

From the table for central concentrated loading (see list), the effective width $=0.69\times16$ feet $=11.04$ feet $=b_c$. Consider the load to be carried by a width of 11.04 feet, use the ordinary formulas for rectangular beam design, and determine the effective depth of the slab, and the area of the steel required. Next determine, by the relation indicated above, the effective width with the load placed 3 feet from the side.

$$b_e=\frac{11.04}{2}+3 \text{ feet}=8.52$$

feet = effective width for eccentric load.

The difference between the value of b_c and b_e is $11.04-852=2.52$ feet. The curb of the parapet should, therefore, be designed so that it will have a resisting moment equal to that of a slab of width 2.52 feet.

Although there are several other conditions which may arise in the investigation of bridge slabs, the few above considered are most important, as they generally control the design.

Waterway for Culverts and Bridges

The calculation of waterway for large bridges is very seldom of importance, as the obstruction occasioned by widely separated piers is very small, and only requires study with reference to the possible scouring velocities around the piers. When short spans are necessary, especially reinforced concrete arches, then the determination of the waterway becomes of great importance, as is also the case for culverts.

The runoff from the watershed of the stream under investigation may be taken from gagings made by the Government, if they extend over a long series of years, but even then phenomenal floods may occur like those in Ohio some years ago, where the Scioto and Miami rivers became veritable torrents. The probable runoff given by the American Railway Engineering Association, for streams in the various sections of the United States in cubic feet per

second per square mile is approximately for the Central, Southeastern, and Southwestern States, 25 cubic feet per second; for the North Pacific Slope 20 cubic feet per second; and for the Northeastern, Mid-Atlantic States, and California 45 cubic feet per second.

There are many formulas for calculating the runoff, and the results obtained from eight well-known ones, for a specific case on the North Pacific Slope where the area of the watershed was 3440 acres or 5.38 square miles, are given in the following list:

	Cubic Feet per Second.
1. Turneaure and Russell	347
2. Burkli-Ziegler's	378
3. McMath	660
4. Murphy	777
5. Fanning	813
6. Kuichling	1455
7. C. B. & Q. Ry.	2120
8. Dickens	2915

The average of all is close to 1200 cubic feet per second, and discarding the first four as abnormally small and the eighth as too great, the average is close to 1300 cubic feet per second. This would seem to indicate that no formula giving less results than Kuichling's should be used, but that for permanent bridges nothing less than the result from the C. B. & Q. Ry. formula should be used, and even that increased as much as 50 per cent for mountain streams or streams where the watershed is being rapidly de-forested. The last four formulas are as follows:

Fanning. $Q=200M^{5/6}$;

Q = discharge, cubic feet per second;

M = watershed area, square miles.

Kuichling. $Q'=\frac{44,000}{M+170}+20$;

Q' = discharge, second-feet per square mile;

M = watershed area, square miles;

C. B. & Q. Ry. $Q=\frac{3000M}{3+2\sqrt{M}}$;

Q = discharge, cubic feet per second;

M = watershed area, square miles.

Dickens. $Q = 825M^{3/4}$;
Q = discharge, cubic feet per second;
M = watershed area, square miles.

The calculations for runoff for a bridge over the Cowlitz river in the North Pacific slope, where the area of watershed was 1560 square miles, disclosed still further evidence that the C. B. & Q. Ry. formula gives as low results as would be at all safe for such a mountain stream, and at a point close to the mountains. The results were as follows:

	Cubic Feet per Second.
C. B. & Q. Ry. formula	56,390
A. R. E. A. value	46,800
Actual rainfall data	62,400

This is taking the A. R. E. A. value at 30 cubic feet per second per square mile, and the actual rainfall at 40 cubic feet per second per square mile, or about 10 per cent increase over the C. B. & Q. Ry. formula. Ultimate safety would be amply provided for in almost every location by adding 50 per cent to the A. R. E. A. values, and the future probably discounted by making a like increase to the value obtained by using the C. B. & Q. formula, or by using the Dickens' formula.

CHAPTER XVII

MASONRY ABUTMENT DESIGN

The design of masonry abutments follows the general method of that given for retaining walls, but where there are wing walls at right angles to the abutment, the pressure on them will be less, owing to the wedge of earth which exerts the pressure being smaller. The vertical reactions on the abutment wall must be taken into consideration as provided in the standards for the Pennsylvania Railroad as given in the following pages. When the abutment has no wing walls and the fill is allowed to surround it, the pressure on the back is counterbalanced to some extent, but it is not advisable to consider the reduction, unless the abutment is very high and very expensive to construct.

There are examples given in this chapter of all of the principal forms or types of abutments, and the designer must select that type which is most suitable and economical for the case in hand. The principal types discussed are as follows:

(*a*) Plain abutments without wing walls.
(*b*) Abutments with angle wing walls.
(*c*) Anchorage abutments.
(*d*) U abutments. (Right-angled wing walls.)
(*e*) Abutments with cantilever wing walls.
(*f*) Box abutments. (Cellular wing walls.)
(*g*) T abutments. (Central wall only.)
(*h*) Pedestal abutments. (No wing walls.)
(*i*) Arch abutments or skewbacks.

The first type (*a*) may be used with economy when there is plenty of ground on which to spread the fill, both at the sides of the approach fill and in front of the abutment.

The second type (*b*) with angled wing walls is most often used where there is plenty of room for the side slopes of the approach fill, but no room in front of the abutment for a slope; or where protection is required from the current in a stream.

The third type (*c*) may be any of the other types, in which the abutment wall is made heavy enough to act as an anchorage for the

anchor arm of a cantilever bridge. The U abutments of the Knoxville cantilever, Fig. 240, were also heavy enough to act as anchorages.

The fourth, fifth, and sixth types of abutments, (*d*), (*e*), and (*f*) are necessary where there is no room for slopes at the sides of the approach fill or in front of the abutment itself. The abutment with cantilever wing walls (*e*), is very desirable when there is no room for slopes and where there is no easy foundation procurable.

The seventh type (*g*) with a tee wall is often desirable when it is necessary to provide for counteracting the pressure or thrust on the abutment, and where there is plenty of room for the side slopes. The tee also carries the track.

The pedestal abutment (*h*) is often suitable for highway bridges where it is desirable to practically avoid the pressure on an abutment, and still provide for slopes at the sides and ends of the approach fill. This form is, moreover, very economical, and can frequently be substituted for some of the other types.

The design of arch abutments or skewbacks (*i*), is always a matter of special study for each particular case. They should usually be designed to harmonize with the structure and in conformity with the principles given in the following pages.

The bridge seats must be made large enough for any of the above types to properly distribute the pressure over the abutment wall for any coexisting conditions of loading. The very least thickness under coping may be assumed at not less than 4 feet, and usually not less than 5 feet.

Pennsylvania Railroad Standards

The methods used by the Pennsylvania Railroad, Alex. C. Shand, Chief Engineer, in designing abutments and abutment piers are given below and the formulas are developed using the notation of Fig. 226. The resultant pressure P must be within the middle third at the base and must be a distance of at least one-fourth b' from the toe at the neat line and base of backwall, or else the wall must be reinforced at the heel so that the resisting moments $a'\Sigma W'+d'Asfs$ is equal to or greater than $2\frac{1}{4}$ times the overturning moment $F'l'$, where As = area of steel, and fs = unit stress in steel or 16,000 pounds per square inch. Reinforcing is recommended at the heel of backwalls, but is not desirable at the neat line.

The greatest eccentricity will often occur without the live load at W_3, in which case the resultant must satisfy the above conditions. Investigation must be made of cantilever toes for direct and diagonal

tension, and they must be reinforced if either one exceeds 40 pounds per square inch. Long low abutments must be investigated for tension in the bottom and reinforced if necessary. Distribution of stress can be limited by construction joints, which must separate

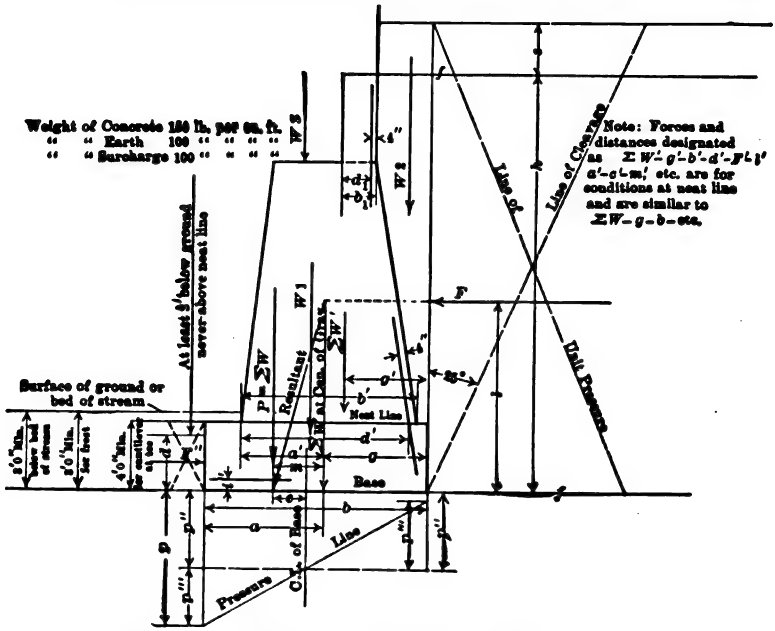


FIG. 226.—DESIGN OF ABUTMENTS. PENNA. RY. CO.

all abutments from wing walls, and the loads at $W3$ must be distributed uniformly over the length of the abutment.

Retaining walls or wing walls of abutments retaining an earth slope, must be figured to resist a surcharge to suit conditions. The fill in front of an abutment is not considered by the Pennsylvania as a force counteracting the force back of the wall.

The area of the face of the foundation below ground shall be such as to resist the force F, with a maximum pressure of 5000 pounds (2.5 tons) per square foot. When the foundations are deep and with firm material in front, the position of the resultant need be only one-sixth b from the toe, provided the sum of the resisting moments $a\Sigma W+(f'''d^2 \div 2)$ is equal to or greater than $2\frac{1}{2}$ times the overturning moment Fl; where $f'''=5000$ pounds ($2\frac{1}{2}$ tons) per square foot, or the maximum pressure on the soil in front; and d = depth of earth in front offering resistance. In any case the permissible toe pressure must not be exceeded.

The forces and distances designated as $\Sigma W'$, g', b', d', Fl', a', e', m', and etc., are for conditions at the neat line, and are similar to ΣW, g, b, etc.

The horizontal pressure on the back for $(h+s)$:

$$F=\tfrac{1}{2}w(h+s)^2 \tan^2 L = 10.87(h+s)^2;$$

for h only:

$$F=10.87(h+s)^2-10.87s^2=10.87(h^2-2hs).$$

The position of application of F:

$$l=\frac{h}{3}\times\frac{35+h}{25+h}.$$

The unit pressure on the back for any $(h+s)$:

$$f=21.74(h+s).$$

The height of surcharge:

s = to 6 feet above base of rail for railway bridges;
s = to 2 feet above surface of road for highway bridges;
s = to 6 feet above base of rail for retaining wall supporting parallel track within line of cleavage.

The height of wall for pressure:

h = from top of backwall to point investigated; and must be taken to bottom of foundation for bearing and overturning at the base.

The pressure F can be reduced for a deep foundation by a couple F with F'', and for a stream F'' can include water pressure from record low water.

The movement of ΣW due to F:

$$m=\frac{Fl}{P}.$$

The eccentricity of P:

$$e=g+m-\frac{b}{2}.$$

The uniform pressure on foundation

$$p''=\frac{P}{b}.$$

The eccentric pressure on foundation:

$$p'''=\frac{6\times P\times e}{b^2}.$$

The total pressure at toe:

$$p=p''+p'''.$$

The total pressure at heel:

$$p=p''-p'''.$$

The following is the formula for the direct calculation of the base (b) with the resultant at $\frac{1}{3}$ point for any depth of foundation (d) with the conditions above the neat line known:

$$b=\frac{\sqrt{d(Fl+\Sigma W'g')+.00444(\Sigma W')^2}-.0666\Sigma W'}{5d}.$$

The value of $\Sigma W'$ and g' include the weight of earth above the foundation offset in the back of the abutment.

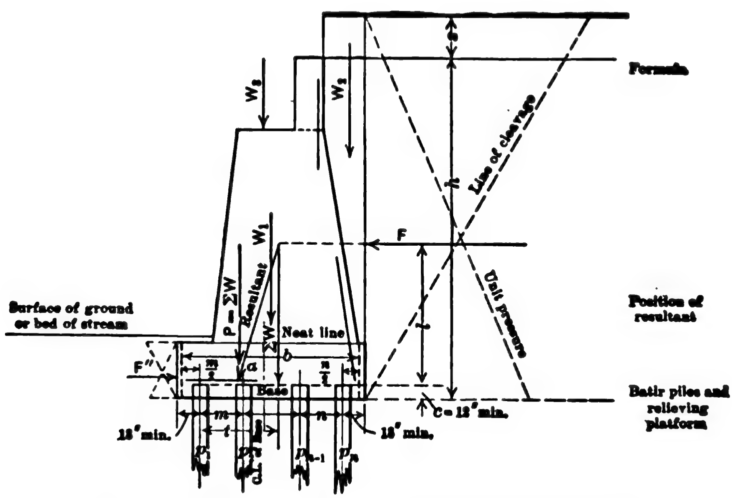


FIG. 227.—DESIGN OF PENNA. RY. ABUTMENTS ON PILES.

When the abutment is to have a pile foundation, Fig. 227, the formulas as just given apply, except as follows:

$$l=\left(\frac{h}{3}\times\frac{35+h}{25+h}\right)-c;$$

$$\Sigma p1 \text{ to } pn=P;$$

$$\Sigma \text{ moments } p1 \text{ to } pn=\text{moment } P, \text{ about any point.}$$

The resultant must be within the middle third of the distance between centers of extreme piles, measured at the pile heads. When the foundations are in non-resistant material, batter piles must be used to resist the force F, and if necessary in extreme cases a relieving or anchor platform must be built behind the abutment.

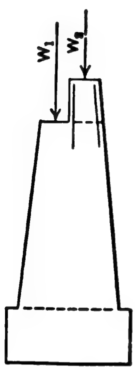


FIG. 228.—DESIGN OF PENNA. RY. ABUTMENT PIERS AND PIERS.

With the center of the front row of piles as a fulcrum, the resisting moment $t\Sigma W$ must be equal to or greater than $2\frac{1}{2}$ times the overturning moment Fl. The determination of the pressure on individual piles makes it necessary to determine the unit pressures on a theoretical base b as shown, and the pressure distributed to the piles in proportion to areas, in order to get resulting pressures p_1 to pn, which will check by moments.

When the construction is for abutment piers, they must be investigated for bearing pressure on the soil or on piles, and for the eccentricity due to the difference between the live loads W_1 and W_2, Fig. 228. They must also be investigated for eccentricity due to the sliding friction of a train, and when the bridge seats are at different levels they must be reinforced as indicated in Fig. 228, to resist rocking or sliding.

TABLE XLII.—CONTENTS OF ABUTMENT PIERS

Approximate Contents in Cubic Yards

Height, Feet.	Total Cu. Yds.		Height, Feet.	Total Cu. Yds.		Height, Feet.	Total Cu. Yds.	
	Single Track.	Double Track.		Single Track.	Double Track.		Single Track.	Double Track.
10	50	95	20	150	300	30	340	650
12	70	125	22	180	350	32	390	740
14	90	160	24	215	410	34	440	835
16	110	200	26	260	485	36	500	950
18	130	250	28	300	565	38	560	1065

Single track, girders 7-foot centers. Double track, girders 7-foot tracks, 13-foot centers. Single track for trusses = double track for girders. Thickness under coping 5 feet, and base = 0.4 total height. Backwall 2 feet thick at top and 5 feet high. Coping 2 feet thick, with 6-inch projection. Footing course 4 feet thick, with 1 foot 6 inch offsets.

Chesapeake and Ohio Northern Ry. Abutments

A steel viaduct, 1080 feet long, on concrete pedestals, was built over the Little Scioto River and the Portsmouth and California Turnpike, on the Chesapeake & Ohio Northern Ry., near Sciotoville, Ohio. There are two bank piers and thirty-four pedestals. On top of these was placed a steel structure with 40- and 80-foot spans, as the approach to the two 775-foot continuous girder bridge spans, designed by Gustav Lindenthal, Cons. Engr. (see Fig. 252).

The first four pedestals and the abutments on the south end were founded on a compact clay with bases spread sufficiently so that the bearing is less than 2 tons per square foot. The next eight pedestals rest on piles driven 15 feet to rock, as it was not considered safe to build on the soil encountered, which was a very wet brown clay. Under each pedestal 16 piles were driven to standstill, on about 3-foot centers. The next eight pedestals were founded on rock, this precaution being taken because they were in or near the river proper. They go into the rock from 1 to 4 feet. Four of these pedestals (or two bents) come in the river, and a cofferdam 24×64 feet in plan was built to take in one of these bents (or two pedestals). A single row of $1\frac{5}{8}$-inch sheetpiles was driven outside 10×12-inch timbers spaced about 4 feet center to center. The water was about 14 feet deep at its maximum, and one pump with a 3-inch discharge easily kept out all the water. The pump was not taxed to capacity at any time. The other four pedestals were put down in the same manner with a 17×17-foot cofferdam inclosing them.

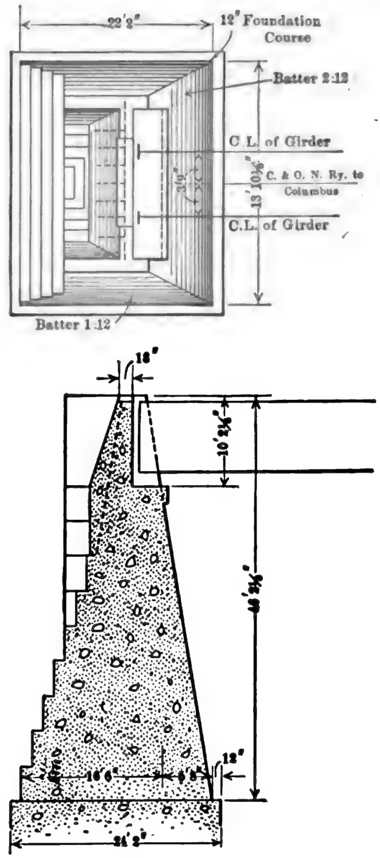


FIG. 229.—CHEASAPEAKE & OHIO NOR. RY. ABUTMENT.

The next four pedestals are on 24-foot piles driven to rock, the next four on 15-foot piles to rock, and the rest are set directly on rock. No serious difficulties were encountered. High water stopped work for only a few days.

In the foundations carried on piles a band of 1-inch twisted steel was placed near the tops of the piles, inside which the vertical reinforcing rods (which run up each of the four corners of a pedestal) were placed and wired together. These bands occurred every 2 feet to the top of the pedestal. The above description is taken from an article by the resident engineer.*

Design of Concrete Abutments without Wing Walls

In designing concrete abutments for railway bridges, there are certain occasions when it is permissible to let the earth fall in front of the abutment: in other words, certain conditions justify construction of abutments without wing-walls. When such is the case, care must be taken to see that the earth does not encroach on the bridge seat.

Assuming that the bridge seat is at some distance above the surface of the ground, the following points enter into the design as important factors, in so far as the question of keeping the earth away from the bridge seat is concerned:

1. Rate of slope of the backfill.
2. Width of the ballast wall proper, and rate of frost batter on back of same.
3. Distance D from the bridge seat to the top of the ballast wall.

In addition to these, the length of the bridge seat as compared with the width of the subgrade or roadbed, and the distance from the subgrade to the top of the ballast wall, also require consideration, the discussion of which, however, will be omitted, as the difficulty arising from these adjusts itself to the circumstances in many cases.

A rock backfill presents but little trouble in this respect as compared with dry, loose earth backfill. This is equivalent to saying that, other factors being the same, the steeper the slope of the backfill, the easier would the solution of the problem be. This can readily be seen by a glance at the accompanying figures, and does not need any further explanation.

The width of the ballast wall is generally taken from 1 foot 6 inches to 2 feet 6 inches for ordinary abutments; and there are conditions which forbid the designer to take excessive width. The frost

* H. B. Watters, in *Engineering News* of January 27, 1916.

batter on the back of the ballast wall can profitably be made light in abutments for through girders, as in that case the distance from bridge seat to ballast wall is comparatively small. Most engineers prefer to have the width of the horizontal section of the ballast wall on the plane of the bridge seat not less than one-half the distance from base of rail to bridge seat. This may be stated by the formula

$$W+Db=\frac{D+a}{2},$$

in which (see Fig. 230) W = width of the ballast wall on top;
D = distance from the bridge seat to the top of the ballast wall;
a = distance from the top of the ballast wall to the base of rail;
b = rate of the frost batter per unit of height.

If this practice is endorsed, ballast walls of abutments for deep deck plate-girders must necessarily be given greater width on top and heavier frost batter on back. The wider the ballast wall and the heavier its frost batter, the further would the earth fall from the bridge seat.

When D is small the problem is comparatively simple: and in fact in many cases a ballast wall of ordinary width with an average frost batter, coupled with slight changes from an ordinary design, is enough to prevent the earth from running on to the bridge seat. This is particularly true in the case of abutment designed for through plate-girders, in which D seldom exceeds 4 feet. Fig. 230 illustrates an abutment of this class. It must be noticed that for through bridges the ballast wall proper is shorter than the bridge seat; so that the earth will start to fall along the side of the abutment at a considerably lower point than it would have done were the ballast wall proper longer than or equal to the bridge seat. Therefore, the excess of the length of the bridge seat over that of the ballast wall proper will of necessity help the matter considerably.

In the case of an abutment for short deck plate-girders, in which D is same as in Fig. 230 but the bridge seat, instead of being longer than the ballast wall, is practically of same length, to attain the desired object the ballast wall is turned at its ends, parallel to the center line of the track, with a battered surface on its back (Fig. 231). In this case the earth begins to fall along the side of the abutment from the level of the subgrade, but at a considerable distance back from the face of the ballast wall.

In cases for long deck plate-girders, however, in which D varies from 7 feet to 12 feet or even more, it is difficult to keep the backfill

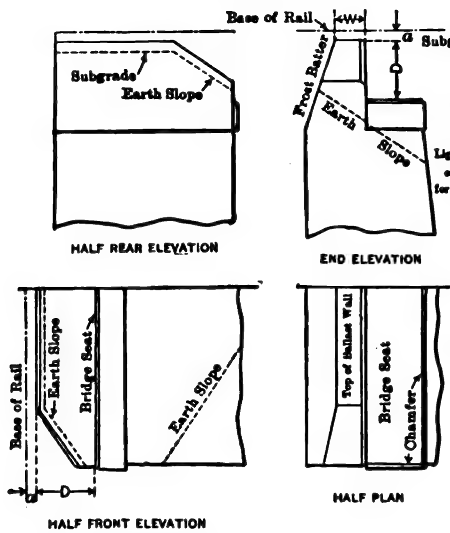


FIG. 230.—CONCRETE ABUTMENTS. LUTHER.

FIG. 231.—CONCRETE ABUTMENTS. LUTHER.

from encroaching on the bridge seat in ordinary abutments, unless short wing-walls are provided. Fig. 232 shows an abutment without wing-walls, intended for deck plate-girders, suitable for cases in which D is about 7 feet or under. To attain the point in view, the ballast wall proper is extended, following the slope of the backfill, and retaining the frost batter of the ballast wall as well, forming a short cantilever. For lack of better terms, we will call these extensions *lugs*. The earth slopes down along the back and to the end of the lug; then turns along its side at some distance below the subgrade, which is determined by the length of the lug, and the slope of the backfill. These lugs are reinforced with light I-beams or old rails. This type of abutment has been extensively used by the Canadian Northern Ry. on their Eastern Lines and has been found very satisfactory.

For same slope of the backfill, the greater the distance D, the longer will the lugs be; so that higher values for D call for excessively long lugs, which are, of course, objectionable. But by modifying the ballast wall, or the lug or both, as the case may require, the matter can be considerably simplified. Fig. 233 shows a design in which D is about 10 feet. In this case the ballast wall is turned at

its ends, parallel to the center line of the track, with a battered surface on the back, and a short lug attached on the front.

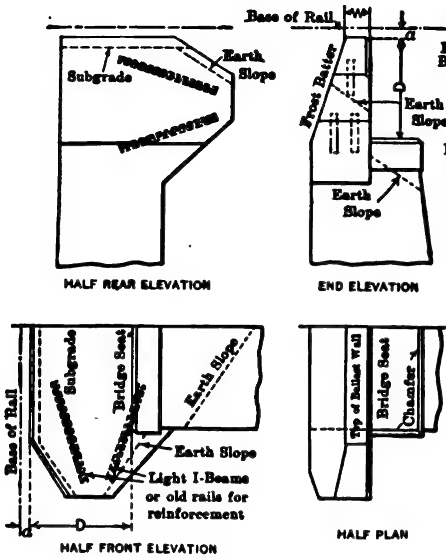


FIG. 232.—CONCRETE ABUTMENTS. LUTHER.

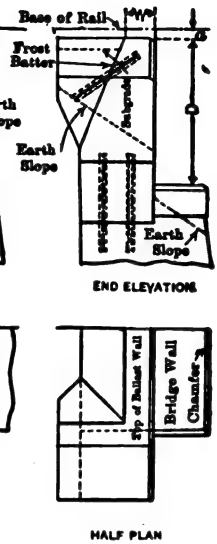


FIG. 233.—CONCRETE ABUTMENTS. LUTHER.

Fig. 234 shows another type with some modifications, having nearly 13 feet for *D*. The ballast wall has been turned and battered on the back, as in the preceding case, while part of the front of the turned portion is vertical. The lug, instead of starting from the top of the ballast wall, starts at some distance below, depending upon the slope of the backfill, length of the turned portion of the ballast wall, and the frost batter of the lug itself.

In these types the ballast wall proper is taken equal to the width of the subgrade, or roadbed. These cases have been cited to serve as simple examples. Special cases must be given special consideration. It is believed that these abutments can profitably be adopted when conditions warrant having the earth partly cover the breast of the substructure. There is, undoubtedly, considerable saving in concrete, in this design, which would not be feasible where short wings are used.

A design is sometimes used similar to that shown in Fig. 233, except that it has little curtain walls on the sides of the bridge seat, extending from the face of the ballast wall toward the breast of the abutment, and following the earth slope. This design, however,

calls for unnecessarily long bridge seat, unless the curtain walls are properly reinforced and then projected beyond the sides of the bridge seat. The vertical surface on the bridge seat may also be objected to, on the ground that undesirable material may accumulate between the girder and the adjacent wall, especially when the bridge seat is comparatively short. Furthermore, the girders during erection may hit the thin curtain walls, and demolish them. The above is taken

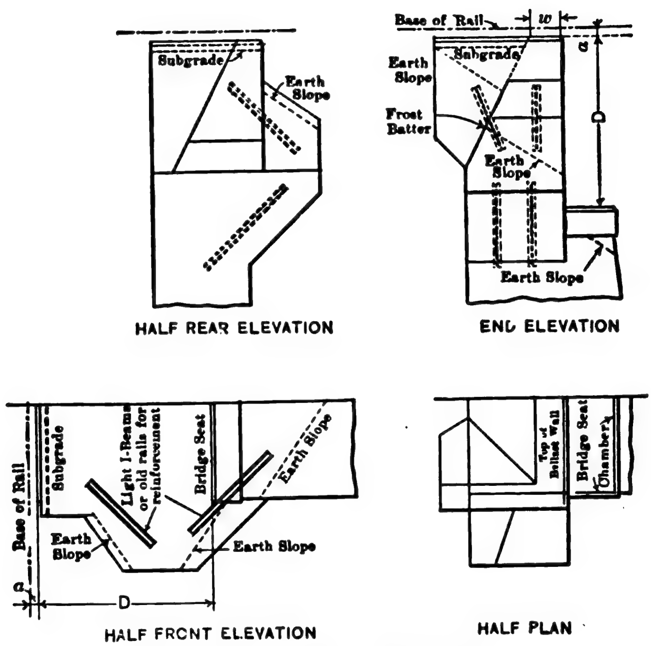


FIG. 234.—ABUTMENT BRIDGE SEATS.

from an article in the *Engineering News* of October 23, 1913, by C. M. Luther, designing engineer.

Anchorage Abutment, Beaver Bridge

The south abutment not only carries the varying loads or uplifts of the anchor arm, but also serves as a retaining wall to a 66-foot embankment. If this pier were to be carried down to rock, its total height to rail level would be about 110 feet, which seemed out of the question, in view of the heavy lateral earth pressure to be resisted by it. The soil overlying rock was quite firm, and was judged easily able to take moderate foundation pressures, at any rate if assisted by

piling. Therefore the decision was reached to found the abutment on a piled foundation a little below ground level. Since this was still some distance above the water plane, concrete piles were planned for.

However, when excavations were made the character of the soil was found to be so good that the pressures of 4 to 5 tons per square

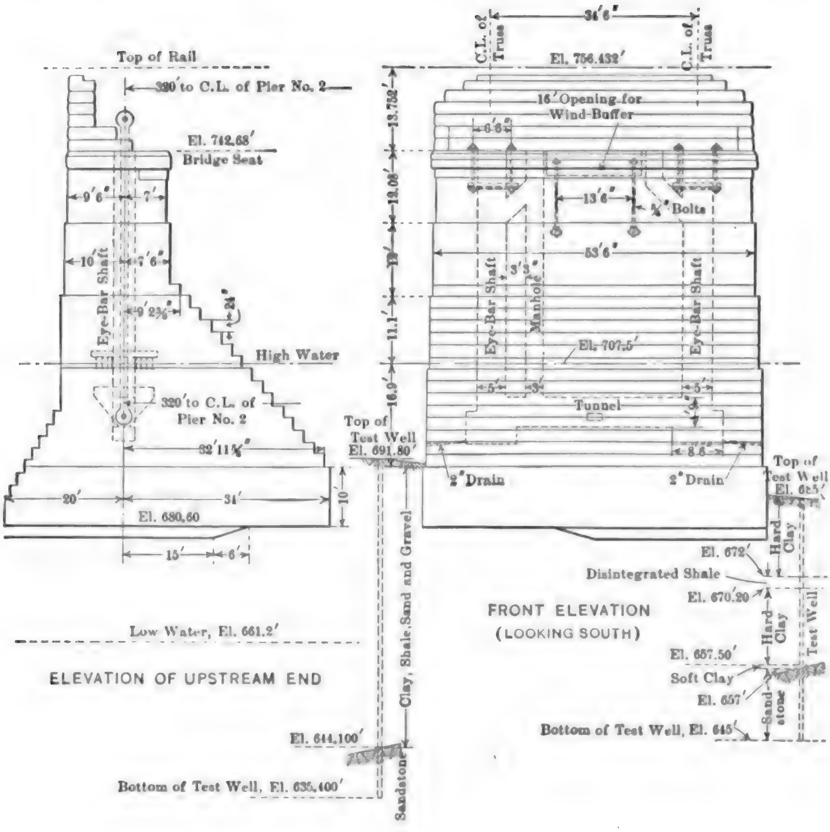


FIG. 235.—SOUTH ANCHORAGE ABUTMENT, BEAVER BRIDGE.

foot which would occur under the base of the abutment could be safely carried by the soil direct. In consequence, the plan of using piles was abandoned, and the abutment was constructed directly on the soil.

The principal elevations of the abutment, Fig. 235, show how the joint service requirements of pier, anchorage and retaining wall were met under the extreme conditions of this case. The total vertical load on the foundation is 13,400 tons, comprising 6570 tons masonry

load, 1800 tons superstructure load and 5030 tons earth load. The pressures per square foot at back and front of base, under different conditions of loading, are given by A. R. Raymer, Assistant Chief Engineer of the P. & L. E. Ry. Co. as follows:

FOUNDATION PRESSURES, SOUTH ABUTMENT

	Tons per Square Foot.	
	Back.	Front.
Masonry unloaded, no fills	3.1	1.3
Masonry unloaded, fill at back	2.5	3.5
Masonry, bridge load, fill at back	3.5	3.5
Masonry, bridge load, fill at back and front	4.2	4.2
Masonry, bridge load, fill back, front and ends	4.0	4.4
Masonry, bridge uplift and all fills	1.9	4.1

As this table shows, the design was so worked out that under full dead-load, as well as when any one of the earth fills is removed, the foundation reaction would be central to the area of the base, in order to give maximum uniformity of settlement. The slope of the earth faces of the wall is adjusted to accomplish this. It is stated that while the embankment fills were being made the abutment settled ½ inch vertically with no horizontal movement.

The means for receiving the anchorage eyebars of the bridge in the abutment are worth noting. Two vertical shafts were cored out in the concrete, connected below by a horizontal passage in which the anchorage girders were set. A third vertical shaft going down to the girder seat was provided as a manhole for use in the work of filling the shafts and passages with concrete later. The design of the end shoes such as to maintain a uniform tension in the anchorage eyebars, this tension being that which occurs under maximum uplift conditions; as this is one of the numerous notable details of the bridge. When the bridge was near its closure point at the center, with the erection traveler at its outer end, producing maximum uplift at the anchorages, wedges provided in the shoes between the upper anchor pin and the shoe base plate were forced in tight, maintaining the eyebars in their stretched condition. Under all conditions of reduced uplift or even resultant downward load, at the anchorages, the tension of the eyebars is absorbed in part or in whole by the masonry of the abutment. This arrangement eliminates all changes of length in the eyebars after the bridge is once completed, and the space around the eyebars can therefore be filled solid without danger that elastic changes of

length of the eyebars will cause a separation between eyebars and concrete and so give opportunity for entrance of water and rusting. The foregoing is taken from an article in *Engineering News* of May 5, 1910. Albert Lucius was Consulting Engineer of the bridge.

TABLE XLIII.—CONTENTS WING WALL ABUTMENTS

New York Central Ry. Standards, Cubic Yards

Height, Feet.	Single Track.	Double Track.	Three Track.	Four Track.
10	170	250	320	385
12	230	325	420	495
14	305	415	520	620
16	390	520	650	750
18	485	640	780	910
20	575	760	920	1070
22	700	900	1095	1255
24	810	1045	1265	1445
26	950	1205	1460	1660
28	1105	1400	1670	1895
30	1285	1610	1905	2175

Wing Abutment Formulas

Laying out a wing abutment, when the layout is to be accurate rather than approximate, calls for some computations that are apt to be perplexing if the proper formulas are not ready at hand. These formulas and their derivation are given in the following, taken from an article by Geo. L. Jensen, in *Engineering News* of November 21, 1912:

Preliminary data in designing a wing abutment are: Width of bridge seat, width of back wall, batters, and footing offsets. The standards of the railroad building the bridge, or the other requirements of the case fix all these quantities except the batters. These latter depend on the height of the abutment, the nature of the soil in the embankment, and the bearing value of the foundation soil. When from these quantities the proper batters have been determined, the remaining problem is to lay out the wing walls. The formulas about to be derived apply to this part of the work.

Assume the conditions to be as shown in Fig. 236. The governing angle of the wing-wall, usually 15°, 30°, or 45° (although there is nothing to prevent its having any other value) is the angle between the base of the wing-wall at the front and a line perpendicular to the center line of the bridge. Let this angle be θ. Then there remain

to be determined the angle ϕ between the top of the wing-wall and the front line of the base, and the angle Δ between the top of the wing-wall and the back of the base.

To illustrate the derivation of the formulas by which these angles ϕ and Δ can be determined, a section of the wing-wall formed by two

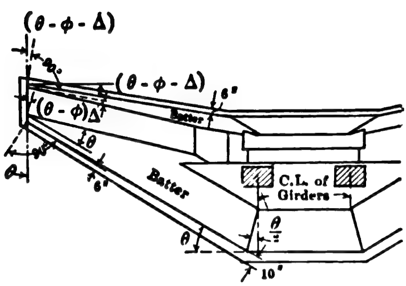


FIG. 236.—WING ABUTMENT FORMULAS.

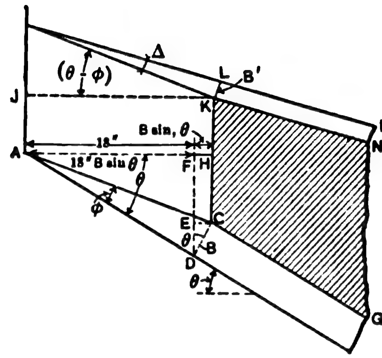


FIG. 237.—WING ABUTMENT. SECTION 1 FT. HIGH.

parallel horizontal planes 1 foot apart will be taken (Fig. 237). The wing-wall top being sloped to match the embankment at a slope of 1 on $1\frac{1}{2}$ in a direction perpendicular to the center line of track, the top face will, in the height of 1 foot, approach the center line of track a distance of 18 inches. But at the same time the face of the wing-wall batters back a distance equal to B inches, and the back of the wing-wall batters toward the face a distance equal to B' inches, where B and B' are the batters in inches per foot of height of the front and back faces, respectively. Then, for the face of the wing-wall, AC in Fig. 237 represents the slope of the front edge of the top of the wing-wall for the 1-foot section, AD the slope of the front edge of any horizontal plane, and CG the slope of the front edge of a horizontal plane lying 1 foot higher. The lines AD and CG are both at an angle θ to the face of the abutment, for they are on parallel sections parallel to the base.

The problem is to find ϕ, the angle between AC and AD. Draw CD perpendicular to AD. Then CD equals the batter B. Draw FD parallel to KC which is parallel to the end of the wing-wall. Then FD is at an angle θ to CD. Then

$$EC = B \sin \theta = FH,$$

and

$$AF = 18 - B \sin \theta.$$

But in triangle AFD,

$$AD=\frac{AF}{\cos\theta}=\frac{18-B\sin\theta}{\cos\theta}.$$

Hence,

$$\tan\phi=\frac{CD}{AD}=B\div\frac{18-B\sin\theta}{\cos\theta}=\frac{B\cos\theta}{18-B\sin\theta},$$

from which ϕ can be found.

For the back of the abutment, the angle Δ is derived as follows: Referring again to Fig. 237, IK represents the back edge of the top of the wing-wall, parallel to the front edge; IM the slope of the back edge of any horizontal plane; and KN the slope of the back edge of a horizontal plane 1 foot higher. The line IK is parallel to AC; therefore,

$$\text{Angle } IKJ=HAC=\theta=\phi.$$

Draw LK perpendicular to IM. Then $LK=B'$, the batter of the back face. Therefore,

$$\sin\Delta=\frac{LK}{IK}=\frac{B'}{IK}.$$

But

$$IK=\frac{18}{\cos(\theta-\phi)}.$$

Therefore,

$$\sin\Delta=B'\div\frac{18}{\cos(\theta-\phi)}=\frac{B'\cos(\theta-\phi)}{18},$$

from which Δ can be determined.

Knowing ϕ and Δ, any of the various angles shown in Fig. 236 can be determined and any of the computations necessary for laying out the abutment can be made.

Reinforced Concrete Abutments

The question of economy in the design of abutments is one that must be very carefully considered in the great majority of cases, and usually the percentage of saving will be found to be about the same as shown in the case of the reinforced concrete retaining walls for the Great Northern Railway in Chapter XVI, and not so great as is shown in the following example of the standard retaining walls of the State of Michigan, and described in an article in the *Engineering News* of

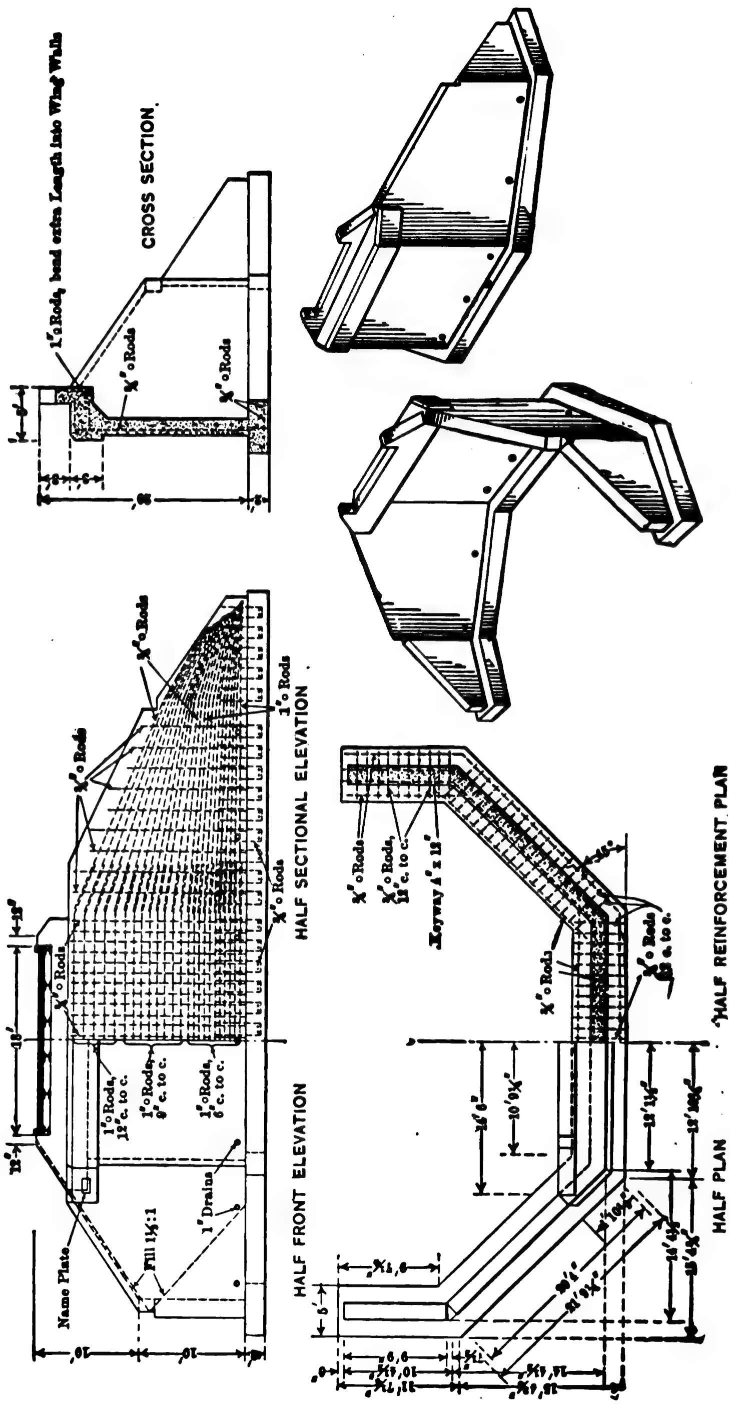


FIG. 238.—STANDARD REINFORCED CONCRETE ABUTMENT, MICHIGAN STATE HIGHWAY DEPARTMENT.

April 20, 1919, by C. V. Dewart, Bridge Engineer of the State Highway Department, at Lansing, Michigan, which is quoted in full.

The economical design of abutments, more especially of those which must retain a great depth of embankment, presents a difficult problem for the engineer. The type of abutment shown in Fig. 238 has been given a thorough trial by the Michigan State Highway Department and found to be a little more satisfactory and more economical than any of the other designs in common use. In highway work more than in any other line it is of the utmost importance to make plans as clear as possible, so that they may be understood by and constructed by the inexperienced contractor, with the least amount of supervision. With this end in view, all battered surfaces have been omitted, and a perspective drawing is shown. The drawing reproduced is a facsimile of the 24 by 30 inch blueprint sent out to contractors, except that the blueprint has no figure dimensions, but is provided with letter dimensions and tabulated quantities and dimensions for walls varying in height from 10 to 20 feet in increases of 1 foot. The drawing represents an abutment to carry reinforced concrete girders. A similar drawing is prepared for abutments carrying steel girders. The tables of dimensions and of reinforcing bars required, also make it possible to show on one sheet in a clear and concise form, everything complete for various abutments ranging in depth from 10 to 20 feet above the top of the footing.

The design embodies an unusual combination of principles in abutment construction, but nevertheless they are all principles that have proved their worth separately. The main wall may be designed as a slab supported on four sides; at the two ends by the counterfort wings, which should make an angle of preferably not less than 45 degrees with the main wall; on the bottom by the footing; and on top by the horizontal girder formed by the bridge seat. The wings may be continued straight on down at any desired slope until they run out at the footing, thus forming a triangular slab supported on two sides. This style of wing is better suited to comparatively low abutments. For high abutments it is usually better when the height is greater than 10 feet to turn the wings back as shown in the figure reproduced, parallel to the roadway, thus limiting the structure to the width of the right of way, providing an additional cutoff wall against scour, and allowing rooms for gutters to take care of the drainage along each side of the embankment. This style of wing forms a quadrilateral supported on three sides by the main wall of the abutment, the footing and the counterfort. The counterfort itself,

although usually covered entirely by the embankment, forms a triangular slab supported on two sides.

The greatest advantage of this design is that the wings are designed to act as counterforts, thereby economizing in concrete as much as is consistent with stability. Practically all other designs embodying this principal make use of a concrete tie between the lower ends of the wings. All such ties, however, when properly designed to resist horizontal shear and vertical bending moment, require considerably more concrete than the counterforts require. The use of ties between the wings has the additional disadvantage of making it necessary to excavate all that part of the roadbed between the ends of the wings, greatly increasing the amount of such work. The great economy of this design over the cantilever abutment is due to the fact that stability is maintained by the general shape of the footing rather than by the width of the footing at any particular section. The advantages of this difference are many and all important. The unequalized toe and heel pressure of the cantilever is entirely done away with. As a rule cantilever abutments require a footing as large and heavy as the corresponding gravity abutment, whereas this design requires a footing to carry the vertical loads plus a relatively small vertical resultant due to the action of the wing buttresses. Further, the abutment as a whole is designed and must act as a unit. The main wall cannot fail without lifting the wings, and the wings cannot crack away from the main wall. All reinforcing bars are placed in the outside face of the wall and therefore in the right place to receive and distribute expansion and contraction due to temperature changes. On the basis of $10 per cubic yard for plain concrete and $15 per cubic yard for reinforced concrete (evidently too low a cost for most locations and for 1920, C. E. F.), this style of abutment will cost practically the same, for 10 feet height, as a gravity abutment providing for the same width of embankment and size of bridge seat; but for greater heights the saving in the reinforced design grows rapidly. At a height of 20 feet above the top of the footing there is a saving of approximately 30 per cent; at a height of 25 feet a saving of 50 per cent; and at a height of 30 feet a saving of 90 per cent over the cost of a gravity abutment.

Standard U Abutments

The necessity for avoiding slopes at bridge abutments by using U abutments has already been mentioned, and an example of a highway abutment of this type is given in the following paragraphs.

The form of similar abutments for railway use is shown in Fig. 239, and they may be constructed of reinforced concrete similar to the abutment shown in Fig. 238, with considerable economy over solid stone or monolithic concrete. This may be estimated roughly from the comparative costs of the Great Northern Railway retaining walls given in the preceding chapter, but for important cases it is usually

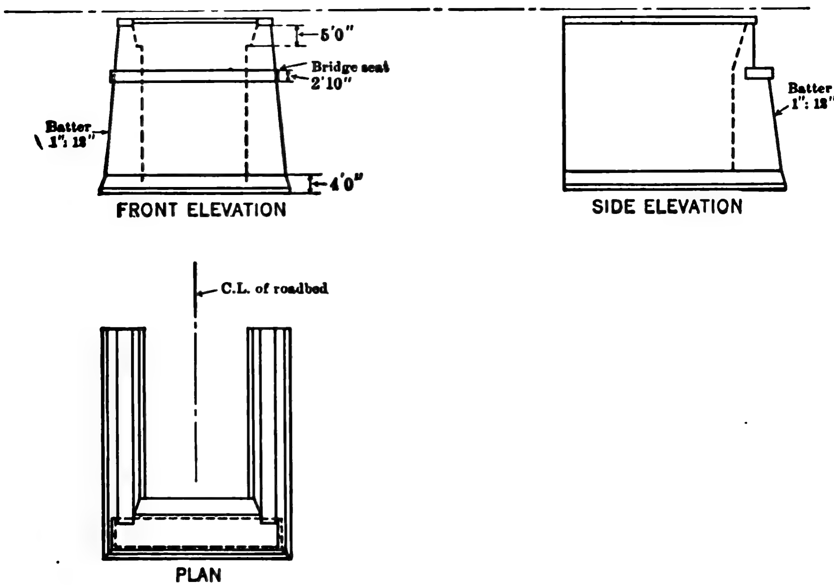


FIG. 239.—TYPICAL U ABUTMENT.

desirable to make comparative designs from dimensions established by careful computations.

Anchorage U Abutments

The abutments of the Knoxville cantilever bridge were of the U type and were constructed by the author of monolithic concrete in the proportions of 1 part of Portland cement, 4 parts of sand, and 7 parts of broken stone.

The one shown in Fig. 240 had a width of 44 feet 4 inches, a height of 17 feet $8\frac{1}{2}$ inches and a 3-foot footing course. The thickness at the base was 0.45 the height or 8 feet as shown. The wing walls were stepped as shown in the plan and were 121 feet $2\frac{3}{4}$ inches long for this abutment. Weep-holes were provided in the locations indicated, 8 by 12 inches in size.

The abutments were also the anchorages for the anchor or shore arms of the adjacent cantilever spans, with four steel beams for

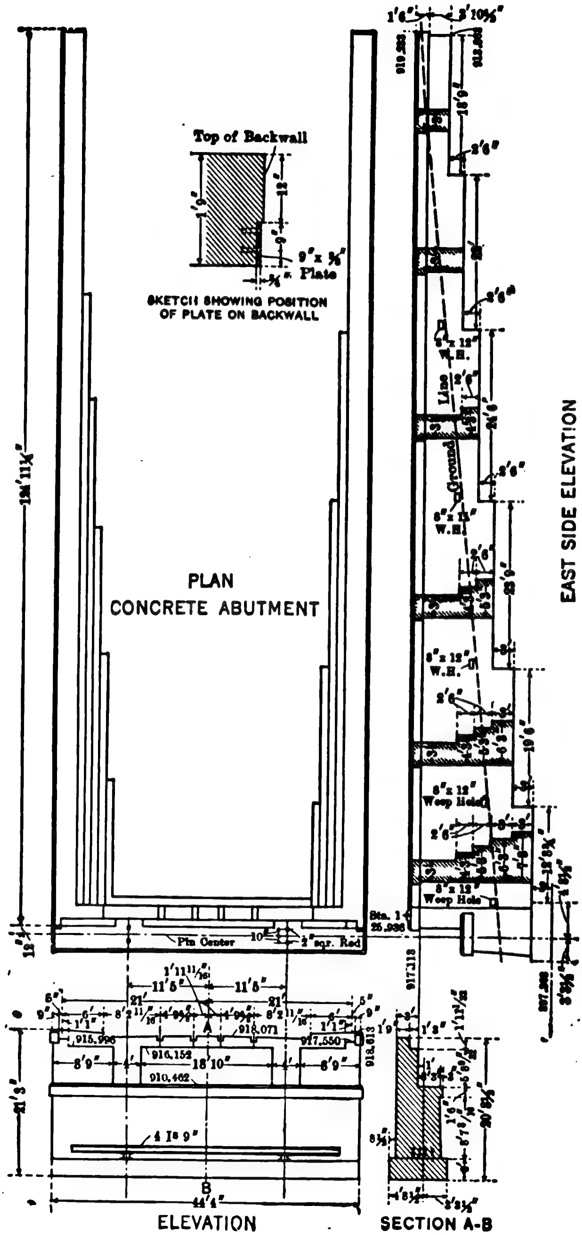


FIG. 240.—KNOXVILLE ANCHORAGE, U ABUTMENTS.

distributing the upward pressure upon the concrete, from each set of anchorage bars.

Bridge Abutment with Cantilever Wings

A special form of concrete bridge abutment was used for bridges crossing the Datteln-Hamm Canal, in western Germany, recently constructed. The canal passes through a coal-mining region, where

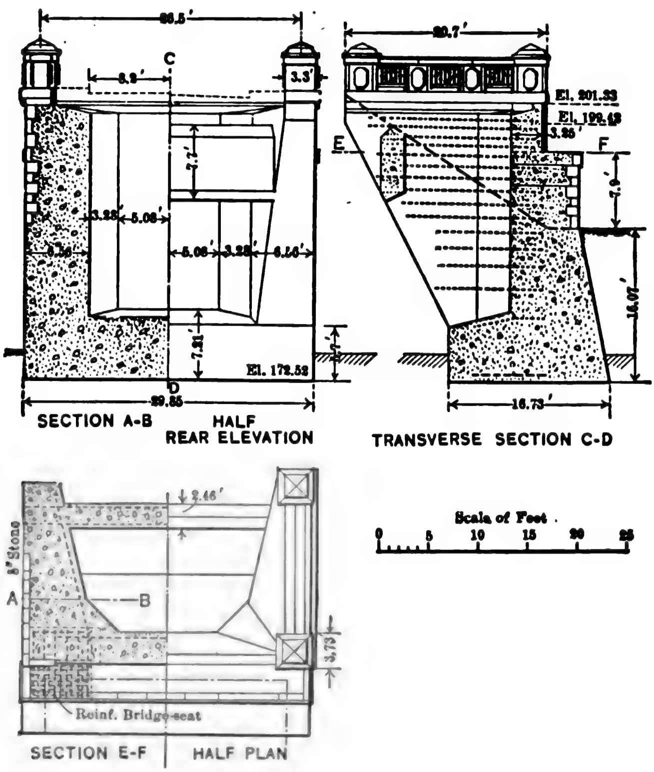


FIG. 241.—CANTILEVER ABUTMENT. CANAL FROM DATTELN TO HAMM.

the ground practically everywhere is subject to subsidences up to a number of feet. While more or less continuous, these subsidences are quite irregular in their distribution along the canal, and also irregular as to time of occurrence. Various special constructions were used in the canal work to provide against injurious or disturbing effect of future subsidences, and among other items the bridge abutments had to be so designed that they would not be in danger of being broken

apart when settlement tends to tilt or shift them. A drawing, Fig. 241, reproduced herewith, shows the type.

The abutment consists of a pier and two rearward wings, which, however, are cantilevered out from the pier and do not rest on bases of their own. As the danger of injury from settlement is about in proportion to the size of the base of the structure, in this case the abutment was made as small as possible in ground plan, that is to say as compact as the allowable soil pressure and other conditions permitted. The whole abutment is made of reinforced concrete. The upwardly sloping wings, which are parallel to each other and to the center line of bridge, are not retaining walls, but act as beams in retaining the earth, being held by the body of the pier from which they project, and at their outer end by a transverse tie. Their inner faces are vertical, so that wedging action of the earth is avoided. Even without this, the outward pressure from earth load and surcharge is such as to require a very considerable cross-section for the transverse tie. The use of reinforced concrete for this member will be seen to make a substantial construction and obviate all rusting trouble. The top surface of the tie is shaped to a ridge, minimizing the earth pressure on it.

The several abutments of this type clearly exhibit the peculiar shape of the abutment and also show how the forms for the concrete work had to be supported under the overhang of the wings.

The exterior or showing faces of these abutments are in general of sandstone, the body being in all cases of concrete. The bridge seats are separately molded reinforced-concrete blocks. Oberbaurat Hermann, of the Königliche Kanalbaudirektion, Essen on Ruhr, was in charge of the canal section to which the illustrations and description pertain. The account is taken from *Engineering News*, Oct. 23, 1913.

Box Abutments

The box abutment constructed on the main line of the Western Pacific Railway at Altamont, Cal., as shown in Fig. 242, represents a very economical design for a combined abutment and retaining wall, and is a modification of the U abutment.

The original structure was a 26-span framed trestle approach to a steel span overhead crossing over the Southern Pacific Railway. The trestle bents averaged about 50 feet in height and were built upon a pile bent foundation. The change in location of a highway passing under the span and the construction of it as a State Highway made a

retaining wall necessary back of the abutment. The soil at this point consists of a stratum of black loam saturated from numerous springs in the vicinity. This stratum is from 15 to 20 feet deep and rests upon solid rock. The original soil was found to be compressed and bulged out laterally by the superimposed fill, until it stood 61 feet

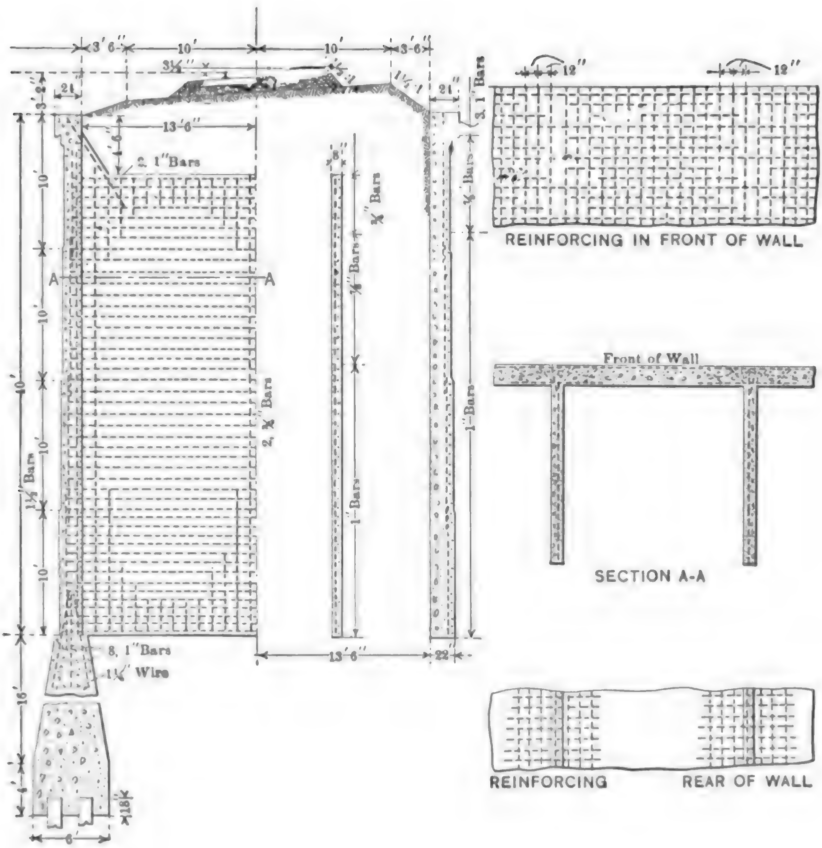


FIG. 242.—BOX ABUTMENT, WESTERN PACIFIC RY.

below subgrade. The rock foundation averaged 76 feet below the track.

An ordinary type of reinforced concrete abutment and retaining wall proved to be entirely out of the question; and as it was desired to fill in the trestle, a type of structure was designed that proved quite economical and allowed construction work to be carried on without in any way interfering with traffic. A slow order of 10 miles per hour had been maintained over the trestle for some time previous to the

construction of the new work, on account of the difficulty of maintaining line and surface under the continuous settlement of the new filling material. This speed restriction for trains was further reduced to 4 miles per hour during the construction of permanent work. The structure as shown in the illustration, consists of a thin abutment tied in by means of reinforcing bars to curtain walls 27 feet apart, parallel with and equidistant from the center line of the track. The curtain walls are held in place by tie walls spaced 15 feet centers, at right angles to the track. The foundations for the superstructure consist of plain concrete pedestals on wooden piles driven to bed rock. The pedestals are 6 feet square at the bottom, 2 feet 6 inches square at the top and 20 feet high. They are placed under the curtain walls at the intersection of the tie walls. The pedestals and the tie walls were placed 15 feet apart longitudinally with the track, in order to fall midway between the trestle bents of the old bridge, and leave them undisturbed during construction of permanent work and completion of filling after the concrete had set.

The curtain walls are 135 feet long, 40 feet high, and have a maximum thickness of 22 inches at the bottom. Each 15-foot span between tie walls is designed as a fixed beam with uniform loading, consisting of the horizontal weight of the enclosed fill plus the weight of the train and impact. Earth filling was assumed at 100 pounds per cubic foot, and for live loading Cooper's E50 loading was used with 80 per cent additional for impact. Working stresses of 16,000 pounds per square inch for tension in steel, and 600 pounds per square inch for compression in concrete were used. Reinforcement consisted of deformed bars rolled from billets of structural steel grade. Concrete was 1 : $2\frac{1}{2}$: 5 mixture. The tie walls are 9 inches thick and simply serve to embed the reinforcement thoroughly, the fill on each side of the wall being assumed to exert equal pressure and to cause no bending moments. This type of structure is therefore a hollow compartment box with neither top nor bottom. It has no overturning moments to contend with, no direct vertical pressures other than the pressure under the foundation pedestals, which is probably very little more than the weight of the concrete itself, or about $3\frac{1}{2}$ tons per square foot. No weep-holes are placed in the curtain walls, as the bottom of the wall on the side opposite to the State Highway stands about 10 feet above the surrounding ground surface. The fill is prevented from flowing out from under the curtain wall by an earth berm 10 feet wide out from, and 6 feet above the bottom of the wall. The cost of the concrete including $2.73 per cubic yard for the reinforcing steel, was $14.12 per cubic yard. The work was done under

the direction of J. H. Knowles, Bridge Engineer of the road, and the account is taken from the *Engineering News.*

Standard T Abutments

The type of construction known as a T abutment is very seldom used, and the form shown in Fig. 243 gives a general idea of the construction. The track is carried on the T in the trough, but the fill will slope at the sides much as in the case of a plain abutment pier. The only two advantages are in the firm support of the track and in the reduction possible in the section of the abutment wall

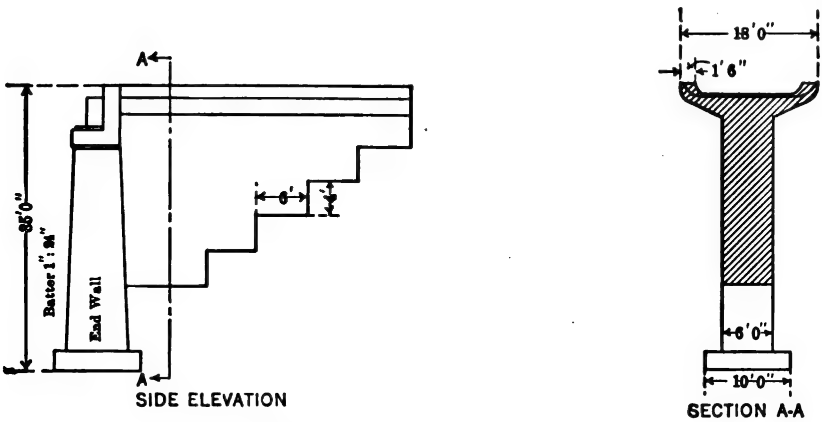


FIG. 243.—TYPICAL T ABUTMENT.

itself. The use of reinforced concrete U or box abutments will usually be found to be more economical.

Concrete Pedestal Bridge Abutments, New York State Barge Canal

In the construction of the Barge Canal across New York State there have been built a number of bridges carrying roads across the line of the canal, either at the sites of old bridges across the smaller but colinear Erie Canal or else at entirely new locations. In either instance it was generally the case that the grade of the road had to be raised considerably in order to reach the bridge level with its required 15-foot head-room, so that the abutments were built high above the original earth surface to retain the filled road afterward placed. For this particular kind of an abutment, the U-shaped type is generally used on most work, but upon the Barge Canal, the

Chief Bridge Designer for the State, Mr. Wm. R. Davis, devised a somewhat novel type, of concrete pedestal and slab abutment, which has been built for a number of bridges across the canal, and which was described in *Engineering News*, 1910.

To suit the varying local conditions the minor details of the design are changed with each bridge, but the general design is the same as that shown in the accompanying drawings and views, which it should, be noted, do not represent the same structure.

The abutment consists of two forward pedestals, under the bridge seat, joined together with a wall high enough up to prevent the side slope from the road fill from the tailing into the adjacent canal, and two rear columns. Connecting the two forward and the two back columns, respectively, are transverse beams which in turn carry the roadway slab, which connects the bridge floor proper to the earth fill approach. This whole structure is of concrete reinforced in places as shown below.

As will be seen, the subsurface here is of sand and clay and so a pile footing under the pedestals and columns was required, but in many of the bridges where a more solid foundation was obtainable, these verticals were run down to rock or hard-pan without the use of the intervening piles. It will also be noticed that the level of the bridge floor is considerably above the surrounding ground and that a fill approach had to be made on both sides.

Fig. 244 shows the detail design of the north abutment of the bridge. In front a wall 9 feet 8 inches long, in the line of the bridge and 7 feet high extends for the whole width of the abutment, resting on a pile footing. Rising from this wall is the slender wall across the abutment to hold the toe of the fill and, on each end, the pedestals which support the bridge seat and one end of the abutment floor slab. The small retaining wall is slightly reinforced, but the pedestals and the main footing wall are of plain concrete. In construction these pedestals are built first and in the top of each one is left a semi-circular or square groove, which is coated with asphalt paint and into which the forward transverse girder is molded. This asphalt-painted groove forms a joint to separate the forward from the rear portions of the abutment.

The remainder of the structure is cast monolithically and is a simple combination of two pile-supported columns, and a heavy slab, crowned to care for the crown in the roadway. Upon this slab the ordinary road is laid.

After the approach has been completed, the side slope tailing down from the fill is paved with large blocks, sometimes in two

slopes as shown in the drawings, Fig. 244, and sometimes in one slope.

From the number of these abutments which have been built along the line of the canal, it has been found that a saving is effected over the old style U-abutment. The forms are easy to build and require only simple lumber and with any kind of skilled concrete

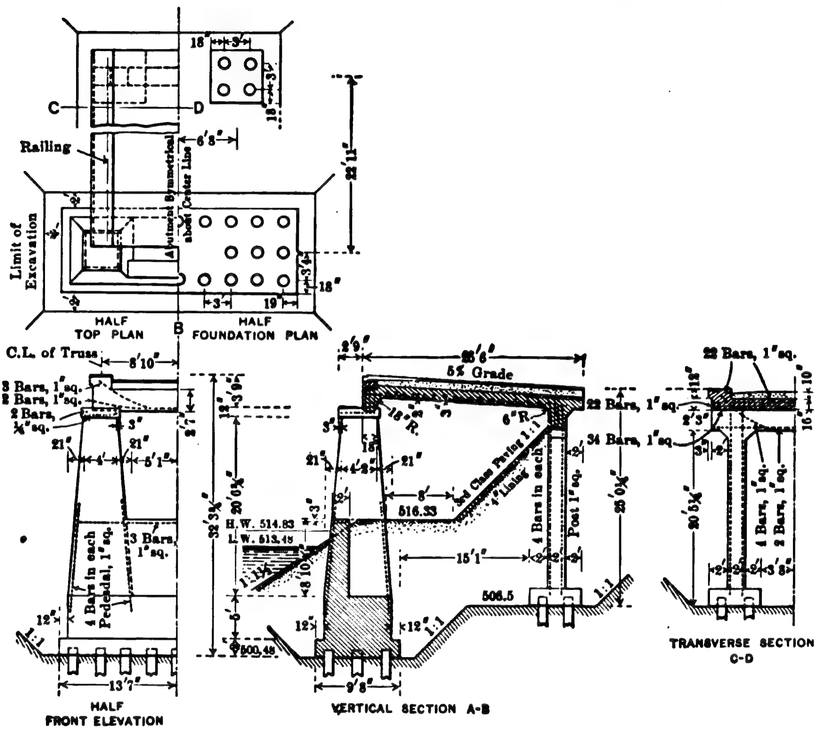


FIG. 244.—PEDESTAL ABUTMENT ON BARGE CANAL.

men and expense per cubic yard is not much greater than in a solid abutment, while at the same time it is obvious that the quantities must be much smaller in the skeleton structure. There has been one objection to some of the abutments that have been built and that is that there is more of a tendency for them to move, if on soft ground, than there is usually found in the solid abutment. It seems probable that this defect could be remedied by sufficient piling, or possibly by a tie back between the two lines of columns. However, in soft ground, the tendency of the lighter construction to move should be looked into thoroughly.

Skewbacks or Arch Abutments

The skewbacks of the Niagara Railway arch of 550-foot span have a thrust to resist of approximately 3000 tons on each bearing. They are built of a special grade of limestone in courses 2 feet thick, with ½-inch joints, and 7 feet 6 inches wide by 15 feet under coping.

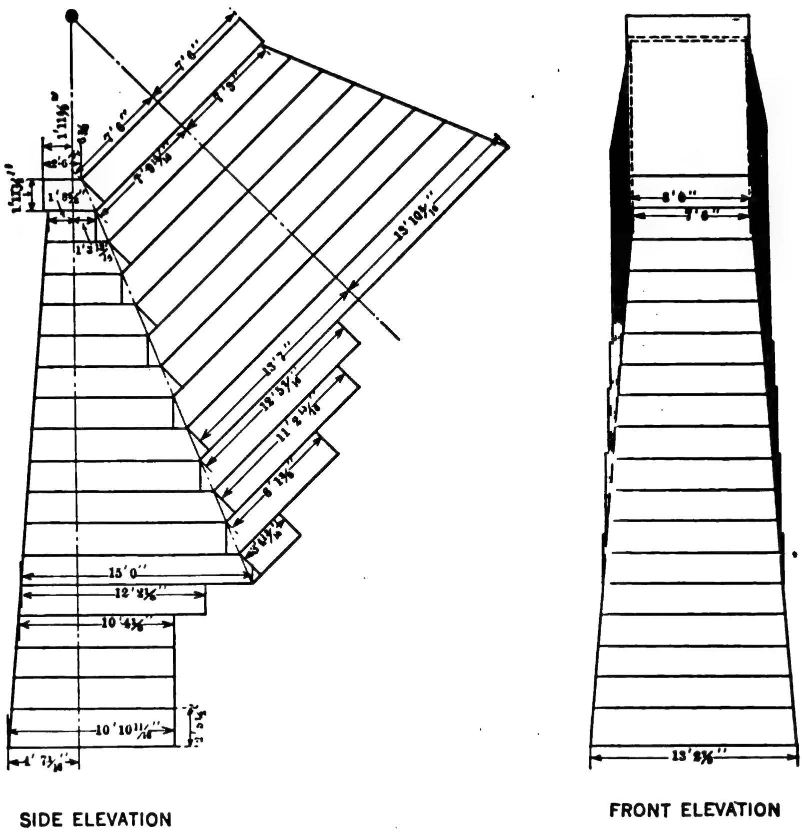
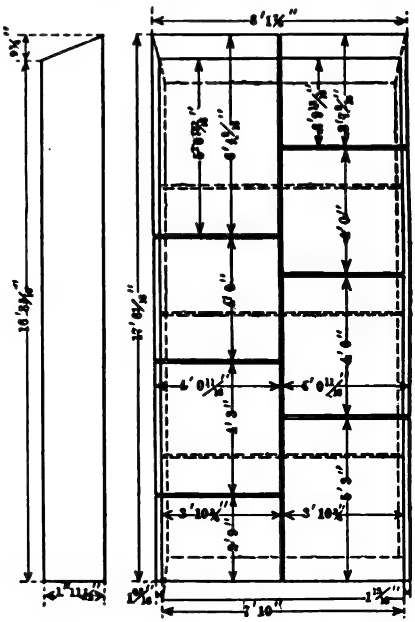


FIG. 245.—NIAGARA RAILWAY ARCH SKEWBACK.

The coping is of granite, with a projection of only 3 inches beyond the limestone. The details of the New York masonry are shown in Figs. 245 and 245*a*, which shows the side elevation, the front elevation, and a plan of the second limestone course. This plan shows the care taken by L. L. Buck, the Consulting Engineer, to obtain a thorough bond between the courses.

These skewbacks were located with a view to bringing the thrust of the arch on the "Clinton Ledge," a solid stratum of gray limestone,

from 12 to 14 feet thick, about half way between the water and the top of the bluff. Above is a blue shale, and below is the beginning of the Medina sandstone formation, thin layers of shale and limestone sometimes running into solid sandstone 4 to 5 feet thick. Borings made near the cantilever bridge just upstream from the arch show that a solid bed of rock is not to be reached at a depth of less than 200 feet. The bearing for the arch comes fairly on the ledge on the New York side, where the stone was cut into at right angles to receive the masonry directly. But on the Canadian side the bearing was not so favorable and concrete had to be used under the front of the south skewback, and under the entire north skewback. The heavy face wall under the New York skewbacks was necessitated by the undue encroachment of the Gorge Electric Road upon the site. The cut made for this road left here an almost vertical face, liable to disintegration upon exposure, directly at the front of the skewbacks.



COURSE - C

Fig. 245a.—Plan of Course C, Niagara Railway Arch Skewback.

All masonry, except the retaining wall under the New York skewbacks, was put in with rented plant and hired labor, with very satisfactory results both as to the cost and the quality of masonry obtained. The maximum loads on the masonry were originally as follows:

	Lbs. per square inch.
On top of coping	339
Under coping	300
On concrete	113

This has been increased within very safe limits by the 1919 revision of the structure for Cooper's E60 train loading.

The rust joint between the masonry and the cast steel shoe, is a mixture of 32 parts of cast-iron filings to 1 part of sal ammoniac by weight, very thoroughly rammed. These ingredients and pro-

portions were adopted after experimenting with several formulas. Thorough ramming is the most important part of the operation.

The view of the skewbacks shown in Fig. 246 was made before the revision of the entire structure by the author in 1919, and 4 feet thrust walls have been introduced between the skewbacks, thus adding greatly to their rigidity and to the appearance of the structure (Fig.

FIG. 246.—NIAGARA SKEWBACKS AS BUILT.

247). The design of similar skewbacks should be carefully studied not simply for strength, but for future increase of loading, and for general appearance. This latter feature will usually cause larger bridge seats to be used, and a very much greater spread of the masonry, or else continuous masonry under both trusses. The data for the foregoing were taken in part from a paper describing this bridge, which was published in the Transactions of the American Society of Civil Engineers for the year 1898.

Skewbacks of Hell Gate Arch Bridge

The massive masonry towers which flank the Hell Gate steel arch greatly enhance the appearance of the bridge and give it its monumental character. They also give expression to the solidity of the abutments to resist the great thrust of the arch. Without the towers,

FIG. 247.—NIAGARA SKEWBACKS REVISED.

the statically trained eye would want that expression of stability, because of the comparative flatness of the shores.

This static requirement, however, is not merely an apparent one. Preliminary wash-borings indicated that the foundations had to go to considerable depth, at least on the Ward's Island side. There having been doubt as to the reliability of the wash-borings, the depth of rock was established later by core-borings at from 55 to 140 feet below mean high water line. To restrict the size of the foundation to a minimum, it was necessary to provide above the ground a mass of masonry, the

weight of which, combined with the inclined reaction of the arch, would give a steep resultant, passing well within the middle third of the foundation area, so that the edge pressure could be kept within permissible limits.

To be expressive of their purpose, the towers were designed architecturally as massive masonry blocks with simple outlines and plain

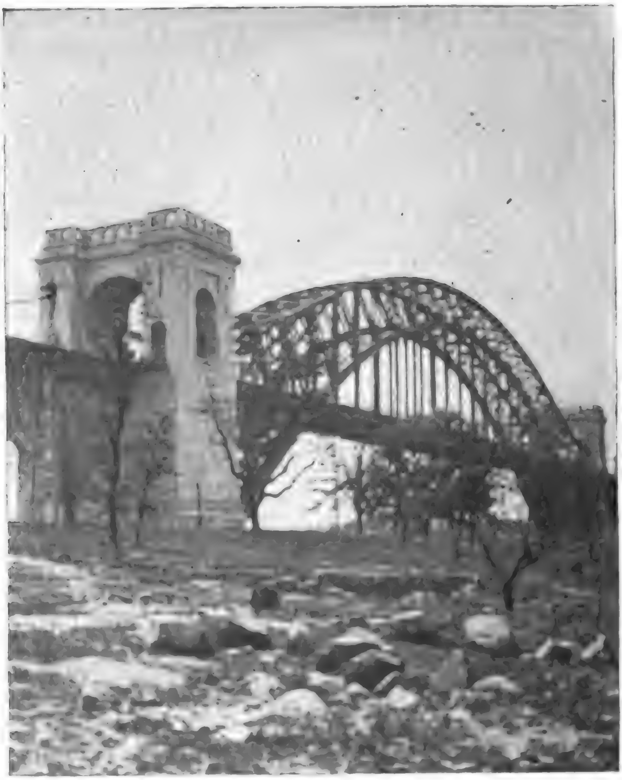


FIG. 248.—HELLGATE ARCH SKEWBACKS.

structural ornamentation (Fig. 248). In working out the architectural form and details of the towers, Mr. Gustav Lindenthal, Consulting Engineer of the bridge, had the valuable assistance and advice of Mr. Henry Hornbostel as Consulting Architect.

Fig. 249 shows the type of construction of the towers above the foundations. The dimensions of the towers are 103 by 139 feet at the ground level and diminish along parabolic lines to 61 by 105 feet at the top. The total height above ground is 220 feet, and the extreme height above bottom of foundation is 345 feet. Each tower

has a solid base, which acts as an abutment for the arch bearings and distributes the pressure over the foundations. On the Long Island side, the base, with the foundation course, forms a monolithic slab, 140 by 104 feet, with an average thickness of 49 feet, and rests on gneiss bed-rock, which was encountered at from 15 to 38 feet below the surface, and was reached by open excavation. The maximum foundation pressure is 8½ tons per square foot. On the Ward's Island side, the base is 140 by 119 feet and 40 feet thick, and rests on the caisson foundation described subsequently.

Above this base, and up to the track floor, the towers are of hollow cellular construction, consisting of the four exterior walls and three interior walls parallel to the tracks. Above the track floor, the transverse walls are pierced by a main arch over the four tracks, and two smaller side arches over the footwalks. The longitudinal walls have also an arch opening for architectural reasons. The towers are topped by a flat roof surrounded by an ornamental balustrade. Stairs lead from an entrance at the ground level through the base and interior vaults to the track floor and roof, and also in the ends of the top chords of the steel arch. The towers are of concrete with granite facing. The concrete is well reinforced with vertical and horizontal steel rods in order to prevent temperature and shrinkage cracks. The track floor and roof are heavily reinforced with steel girders.

On the Ward's Island side, the tower base rests on twenty-one concrete caissons, all sunk by the pneumatic process to depths varying from 37 to 107 feet below mean high water, which is 20 feet below the ground surface. The caissons are arranged in five rows parallel to the axis of the bridge. Each outer row and the middle one consists of five cylindrical caissons, 18 feet in diameter, which are calculated to carry only vertical pressure. Each of the two other rows consists of three rectangular caissons, 30 by 41 feet, which are interlocked by concrete keys extending nearly the full depth of the caissons. These two rows of caissons thus form two rectangular blocks, 30 by 125 feet, which are calculated to resist entirely the horizontal pressure from the arch. They exert a maximum edge pressure on the rock foundation of 20 tons per square foot if skin friction and buoyancy are neglected. The space for the keys was excavated and filled with concrete, partly with and partly without the use of air pressure, after the caissons had been sunk to their final depth.

The dissection of the foundation into twenty-one individual caissons was advisable on account of the large size of the tower base and the greatly varying depth to solid rock. The formation of the bed-rock below Ward's Island is very peculiar. One of the lines

of cleavage between the dolomitic limestone formation and the gneiss rock which run parallel with the East River, appears to pass right under the Ward's Island tower. About 45 per cent of the foundation on the river side is on limestone and about 25 per cent on the land side is on gneiss. Between these two rock formations there is a crevasse of unknown depth and of from 15 to 60 feet width. Its sides are almost vertical, corresponding to the vertical stratification of the rock. The crevasse is filled mostly with red clay and some boulders. The clay is practically impervious to water, which is shown by the fact that the excavation was in part carried on with an air pressure of only 18 pounds at a depth of 100 feet below water level, and, for some caissons, to 18 feet below the cutting edge. In its natural state, the clay is very hard and has a high bearing capacity, but, in water, it dissolves rapidly.

Three of the fifteen cylinder caissons rest entirely on this clay, at depths of from 94 to 123 feet below the surface, where there is no danger of disturbance. Under the rectangular caissons, the crevasse was bridged over by a concrete arch which was built at considerable risk below the cutting edge of the caissons. For this purpose, the caissons were supported temporarily by pilasters of rubble masonry built down to the intrados of the arch previous to the excavation for the latter. All caissons are of 1 : 2 : 4 concrete, well reinforced horizontally and vertically with steel rods.

Owing to the unusual and difficult character of the Ward's Island foundation, the Company decided to do the work with its own forces, under the direction of the Chief Engineer.

The two towers contain approximately 110,000 cubic yards of masonry, of which 28,000 cubic yards are in the Ward's Island foundation. The principal quantities are:

Concrete (1 : 2 : 4 and 1 : $2\frac{1}{2}$: 5)	99,000 cu. yds.
Granite	11,000 cu. yds.
Steel reinforcement	1,000 tons
Structural steel	500 tons

The above account is taken from a paper by O. H. Ammann, Principal Assistant Engineer of the structure, in the *Transactions of the American Society of Civil Engineers for* 1917.

CHAPTER XVIII

DESIGN OF PIER MASONRY

THE calculation of bridge piers has been discussed in Chapter XIV, and the data given in Chapters XII and XIII as to their location and the cross-section of the shaft, forms the basis from which to begin the design of the masonry. The fact has been clearly established that the cross-section for ordinary stone or concrete piers in a rapid current should be such as is shown at (*e*) in Fig. 176, but there is no reason why a concrete pier and especially a reinforced concrete one, should not be built of elliptical cross-section as shown at (*f*) Fig. 176. There are many cases where piers of this latter cross-section could be constructed of a hollow reinforced concrete shell, have sufficient weight to withstand the forces acting on the pier, save a large amount of weight on the footings and foundation, and by means of a reinforced concrete slab coping, carry the bridge superstructure in an entirely satisfactory manner.

Classes of Stone Masonry

The masonry of stone piers may be either, ashlar, squared stone range work, broken range work, or rubble; but for first-class work it should be either ashlar or squared stone range work. The specifications of the Pennsylvania Railroad may be taken as the standard for the four classes, and the following brief synopsis of it gives the essential features.

Ashlar masonry consists of squared or cut stone blocks with rectangular dimensions, and holding full size throughout.

Squared stone range masonry consists of work laid in regular courses set in cement. The courses to be not less than 14 inches thick and not more than 30 inches and to diminish regularly from bottom to top. The joints in face stone to be squared to the face for a depth equal to at least two-thirds of the thickness of the course, but for the thinnest courses not less than a 12-inch bed. Stretchers to be not less than 4 feet and not more than 8 feet, but shall have a bed at least one-fourth greater than the thickness of the course. Headers to be not less than 4 feet in length, and shall constitute not less than

one-fifth of the wall; no header shall have less than 18 inches width of face. The backing may be of large sized, well shaped stone, or of concrete. The coping of such masonry to be of cut stone ashlar.

Broken range masonry consists of the same class as squared stone range, except the face of the wall is not in continuous courses, but broken up by stones of different thickness. No stone to be less than 12 inches thick, nor more than 24 inches, and no stone shall have less bed than its thickness. All joints shall be vertical and horizontal.

Rubble work consists of stone of about 6 cubic feet each, laid in regular courses so as to make firm and compact work. No stone shall be less than 2 cubic feet, except for filling interstices between the blocks in the heart of the wall. At least one-fifth of the wall shall be headers.

The data as to the analysis of Pennsylvania standard piers is given in Ch. XVII, but for making a complete investigation as to strength and stability the formulas and data referred to, and given in previous chapters, should be consulted and used.

Fundamentals of Pier Design

The conclusions given in the Bulletin of the American Railway Engineering Association for February, 1918, are quoted in the following paragraphs, as showing their agreement with the fundamentals given in this volume and as elaborated in Appendix XI.

(1) That for important structures wash borings as a means of determining character and bearing values of foundation soils are not generally reliable.

The most commonly known method of making wash borings is by means of a double pipe where the material is washed out by a stream of water through the center pipe as the outer pipe casing is driven.

Samples of the soil are obtained by catching at frequent intervals samples of the washings, and separating the sediment by settlement.

Another method quite commonly used where hard material is encountered is the percussion drill, which pulverizes the materials it passes through. The pulverized material is either removed by a sand pump or washed out with a center pipe.

In either method the materials are thoroughly churned. Correct interpretation of the character of the soil by these methods is doubtful, depending as it does on observation of the speed of driving the casing and the drilling; furthermore, in the case of wash borings it is impracticable to distinguish solid rock from boulders.

That for important structures core borings give the most reliable

data. The borings should in general be carried at least 10 feet into rock, when encountered.

Diamond, calyx or similar drills furnish cylindrical cores of all strata of hard material. Soil, sand, clay and lighter materials will be brought to the surface similar to wash borings.

(2) That for important structures, where there is no reliable data, or where there is any question of the safe bearing value, soil bearing tests be made; the test loads being increased until settlement occurs or until twice the bearing load it is proposed to use in the design has been reached.

That one-half the ultimate load thus found be used in designing the foundation except that three-quarters of the ultimate load may be used for maximum toe pressures produced by tractive force or wind; provided, however, that the safe load thus found does not exceed the safe crushing value of the materials of the substructure.

By ultimate load is meant that load that can be placed on the soil without undue settlement. It is not intended to consider a slight compression of the soil from $\frac{1}{4}$ to $\frac{1}{2}$ inch as settlement except in the case where the settlement is progressive under the load applied.

(3) That pile bearing formulas based on the fall and weight of the hammer are not always a true index of the safe bearing value of the pile, but are of value in determining the extent to which driving is necessary in a soil of known resistance.

Pile-bearing formulas are not reliable in the case of the softer soils, where the carrying power is determined by the surface friction of the pile and there is a large increase in the supporting power of the pile after the driving ceases and the pile is allowed to remain unloaded for several hours, nor are they reliable in the case of piles driven through soft soil to rock or other hard material.

That for important structures where data is lacking or where there is any question of the safe bearing value, load tests be made; the loading being increased until settlement occurs or until twice the load per pile it is proposed to use in the design is reached.

That one-half the ultimate load thus found be used in designing the foundations except that three-quarters of the ultimate load may be used for the maximum pressures produced by tractive force and wind; provided, however, that the safe bearing value of the piling or materials of the substructure are not exceeded.

By ultimate load is meant that load that can be placed on the pile without undue settlement. It is not intended to consider a slight depression of the pile from $\frac{1}{4}$ to $\frac{1}{2}$ inch, as settlement except in the case where the settlement is progressive under the load applied.

In arriving at the safe bearing value of piles consideration must be given to their action as columns taking into account the supporting action of the penetrated material.

(4) That in general, buoyancy of structures in water be not considered as reducing the foundation load except in the case where water has free access to base of the foundation or when calculating stability from overturning.

(5) That the loads to be considered in designing foundations are the total dead loads of substructure and superstructure, the live load, including an allowance for impact, tractive force, wind and ice pressure, and earth pressures in the case of retaining walls and abutments.

(6) That in general, the cut-off of wood foundation piles in tidal waters shall not be above mean daily tides and shall not exceed 2 feet above mean low water.

That in general, timber in foundations in tidal waters shall not be used above mean daily tides nor shall it be used at a greater elevation than 2 feet above mean low water.

That in general, wood foundation piling and timbers be kept entirely below the probable lowest ground water level, except in the case of tidal waters as above noted.

That in all waters where marine borers exist, no untreated timbers should be used above the permanent mud line of the bottom.

(7) That the spacing of wooden piling in foundations be not less than 2 feet 6 inches, center to center.

(8) That the bottom of foundations should be placed entirely below the line of frost action, the depth of foundation depending on the locality.

(9) That the calculations of foundation pressures be made in accordance with the rules and formula for the design of retaining walls published in the Supplement to the Manual, Vol. 19, No. 197, pp. 47–55.

The piers illustrated in preceding chapters such as those for the Bismarck bridge, Fig. 160; the first Memphis bridge, Fig. 163; the Omaha bridge, Fig. 174; the Russian State Railway pier, Fig. 175; and those for Knoxville cantilever, represent first-class design and construction of piers for heavy bridges; and in the following pages will be given data and dimensions for piers of this type; also for lighter masonry piers, and for reinforced concrete piers of various designs.

Light Highway Piers

The piers for ordinary highway bridges when of stone or monolithic concrete, are usually square ended or of very simple design, as shown respectively in Fig. 20, a photograph of the old Howe truss bridge at Knoxville, Tenn., and in Fig. 250 which is Cooper's standard type of highway bridge piers, although a better cross-section than either should be employed where there is considerable current.

TABLE XLIV.—CONTENTS PLAIN BRIDGE PIERS

Rectangular. Same Batter on all Faces. Size is Under Coping. Contents Cubic Yards

Height.	5 by 20 Feet.		6 by 22 Feet.		6 by 34 Feet.	
	Batter 1 to 12.	Batter 1 to 24.	Batter 1 to 12.	Batter 1 to 24.	Batter 1 to 12.	Batter 1 to 24.
5	20	20	25	25	40	40
10	45	40	60	55	90	80
15	75	65	95	85	140	130
20	110	90	135	115	205	175
25	145	120	180	150	270	230
30	190	150	235	190	350	285
35	240	180	295	225	440	340
40	295	215	355	270	525	405
45	350	250	425	315	625	475
50	420	290	505	365	730	545
55	490	335	595	415	825	615
60	575	380	680	470	970	695

Cooper's standards are of rectangular cross-section, with a batter of 1 to 24, and with a triangular cut water or ice breaker. They are nominally designed for two footing courses, with the usual offsets, although this would be varied to suit a foundation bed on rock or one on piles. The same general sizes with a semicircular end downstream, and an end with intersecting arcs upstream would be preferable and all of the general and tabulated data apply very closely. The quantities for different length spans, for different heights, and for various classes, including single- and double-track electric railway bridges, are given in the accompanying Table XLV.

New Memphis Cantilever Bridge Piers

The new cantilever bridge at Memphis, Tenn., built for the Rock Island and the Gould Lines, by Ralph Modjeski, Consulting Engineer, has piers closely of the Morison type as described in Chapter

TABLE XLV.—PLAIN PIERS WITH CUTWATERS

Approximate Contents in Cubic Yards one Pier

Span in Feet.	Roadway.	Height Pier Top Coping to Bottom Footing.				
		10	15	20	25	30
100......	12 feet........	29	44	60	77	94
	20 feet........	38	59	82	108	136
	Single track....	31	46	62	80	100
	Double track ..	50	75	102	132	166
150......	12 feet........	34	51	70	90	111
	20 feet........	46	70	95	125	157
	Single track....	37	54	74	96	120
	Double track...	38	80	118	153	191
200......	12 feet........	39	58	80	103	128
	20 feet........	53	80	109	143	178
	Single track....	43	63	86	112	140
	Double track...	66	99	135	174	217
250......	12 feet........	44	66	90	116	145
	20 feet........	61	91	123	160	199
	Single track. . .	48	74	98	127	159
	Double track...	73	109	149	192	238
300......	12 feet........	49	73	100	130	162
	20 feet........	68	101	137	177	220
	Single track....	54	80	109	142	178
	Double track...	80	120	164	210	260

300-foot spans should be not less than 16 ft. min. clear width.

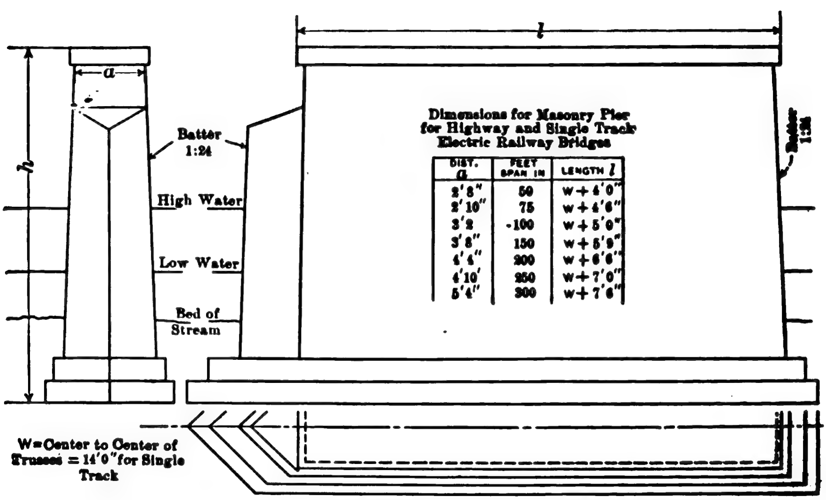


FIG. 250.—COOPER'S HIGHWAY PIERS.

XII and as shown in detail in Fig. 253. The location of the piers was fixed by the necessity for placing them exactly downstream from those of the old bridge, which was only 200 feet upstream. The

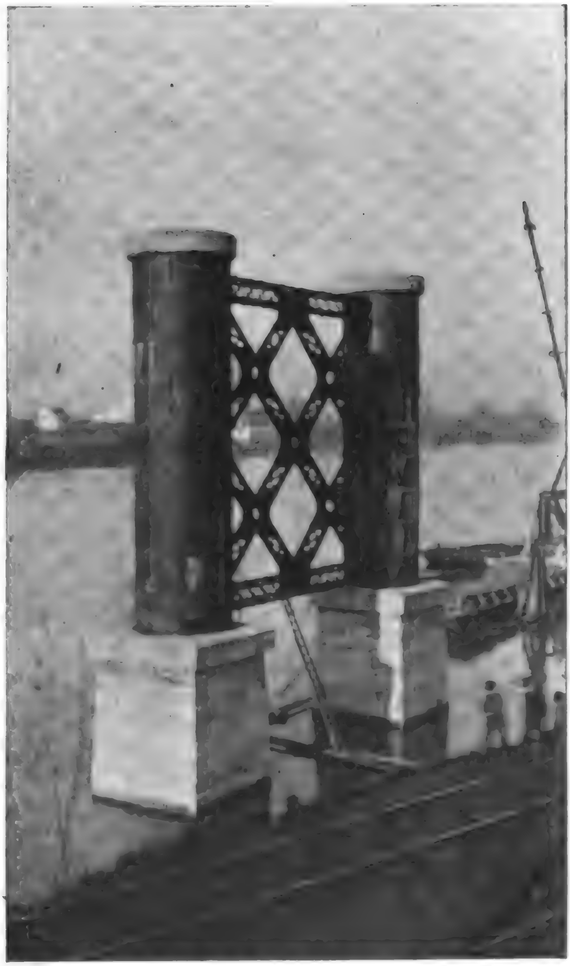


FIG. 251.—COMPOSITE HIGHWAY PIER

five piers were sunk by the pneumatic process to depths of from 70 to 110 feet, and are of concrete faced with granite from Stone Mountain, Georgia.

The main piers are 14 feet thick under the coping, and 54 feet long, including the semicircular ends. The faces are battered one

in twenty-four, which make the base of the tall shafts about 25 feet by 75 feet. The coping and corbel course from a cornice of 3 feet 6 inches thickness at the top of the pier and the offset from the circular ends to the pointed ends, just above high water, adds to the appearance of the design, and the continued use of this form is justified not only by its architectural appearance, but by its simplicity,

FIG. 252.—CHANNEL PIER, SCIOTOVILLE BRIDGE.

its correct cross-section in the current, and its general fitness for the functions it has to fulfill.

McKinley Bridge Channel Piers

The piers of the McKinley bridge, over the Mississippi River at St. Louis, are of the same type as those for the new Memphis bridge, and are shown in Figs. 254 and 254*a* and *b*, being designed and constructed under the same engineer. The account taken from the *Engineering News* is particularly valuable on account of giving

extensive data as to river conditions, borings, and the surveys for locating the piers.

The Mississippi River at the site of the McKinley bridge is about 1950 feet wide at low water stage and nearly double this width at high water. The west

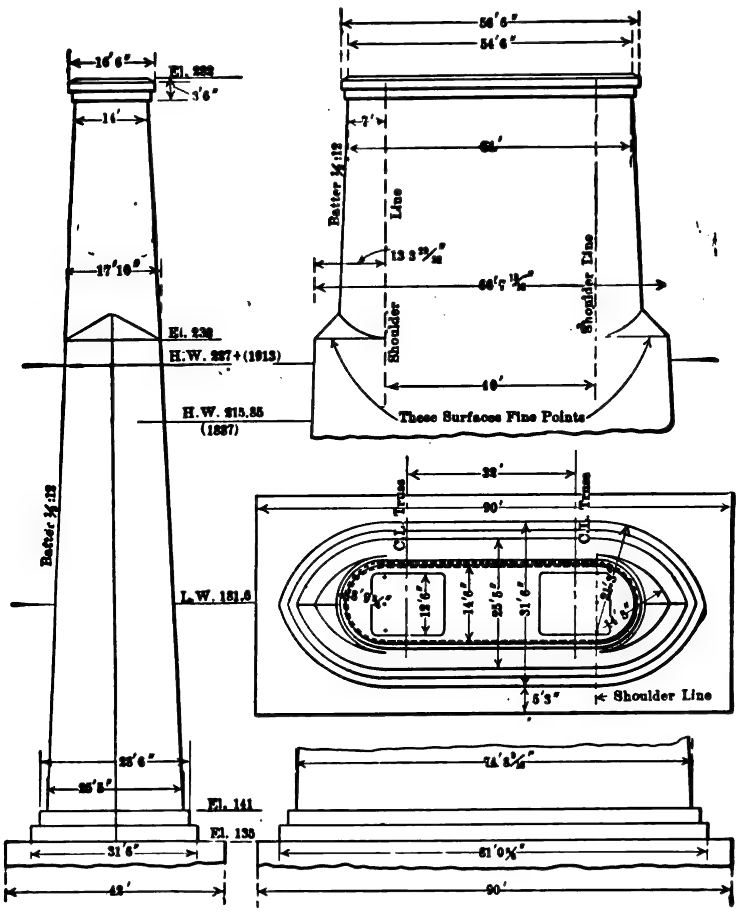


Fig. 253.—Memphis Bridge, Main Piers.

bank has the steeper incline, beginning at low-water shore line near the west inner harbor line and reaching above the high water line at the embankment of the C., B. & Q. Ry. in a distance of 300 feet. The east bank rises with a gradual slope to an elevation of about 15 feet below the high water line near the east inner harbor line.

The river bed is composed of sand which is continually changing its level, but in general its slope is toward the east. For a distance of 600 feet east of

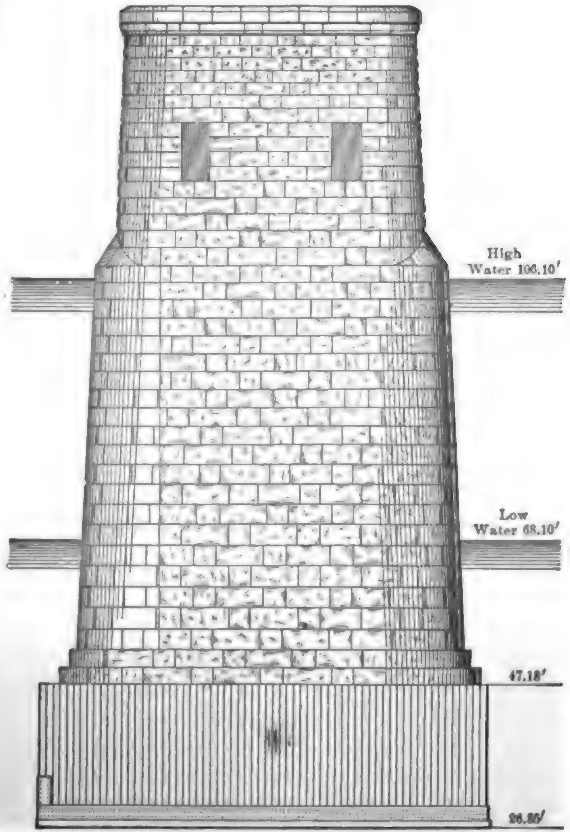


FIG. 254.—MCKINLEY BRIDGE CHANNEL PIERS.

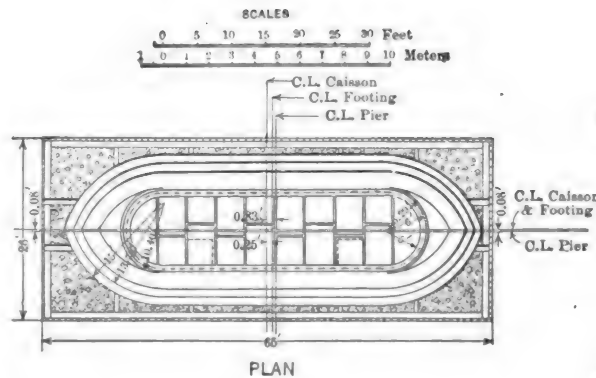


FIG. 254*b*.—SECTION AND PLAN, MCKINLEY BRIDGE PIERS.

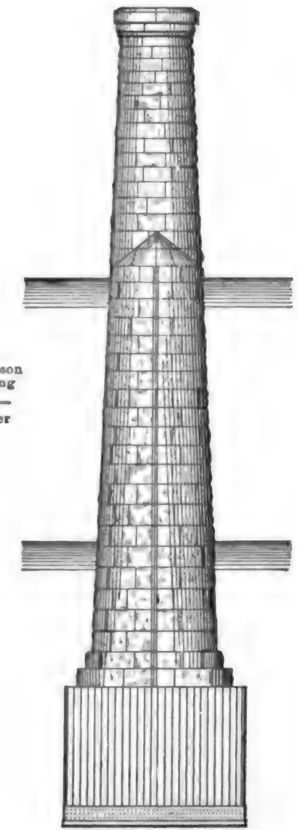


FIG. 254*a*—MCKINLEY BRIDGE CHANNEL PIERS. END ELEVATION.

the west inner harbor line it is comparatively level at an elevation of from 1 to 4 feet below the low water line. It then slopes gradually to the east, reaching its lowest point about 400 feet west of the east outer harbor line, where it is from 18 to 25 feet below the low water line. The changes noted in the river bed during the construction of the bridge were such as to transfer the natural channel of the river towards the east.

The location of the piers was governed by the harbor lines adopted by the War Department in 1903. The distance is 1500 feet between the outer harbor lines and 2000 feet between the inner lines, thus making a levee area 250 feet wide on each side of the river. The plans approved by the War Department provided for clear spans of not less than 500 feet at low water between the outer harbor lines. This required the location of the outer piers outside of these lines, and the lengths of the main spans were made as above stated. The difference in distance between centers of piers is on account of the larger size of the inner piers. The actual distance from the outer harbor line to the center of the adjacent pier was made 32 feet 6 inches.

The War Department required a clearness of 50 feet between high water and the lowest point of the superstructure. The assumed high water level is that of June, 1903, when a stage of 38 feet was reached; this was the highest stage since 1844, when 41.32 feet was recorded by the Mississippi River Commission. All elevations used on the bridge were referred to a datum of 100 feet below the St. Louis directrix which marks the elevation of high water in 1828. Check levels were run over the Merchants Bridge between St. Louis city bench marks and the U. S. bench-mark No. 8 established in Venice, Ill., from the Memphis datum. These levels were checked across the new bridge piers as soon as construction permitted. The low water stage, that of 1863, coincides with the zero of the St. Louis gage, which is 33.7 feet below the city directrix, or at an elevation of 66.3 feet referred to the bridge datum. The elevation of assumed high water was therefore 106.1, allowing for a slope of the water surface of 0.6 feet per mile.

Probably on account of the banking up of the water at the Eads and Merchants' bridges, the slope of the water surface between these structures varies with the rising or falling stage of the river for stages of 20 feet and above. During the construction of the McKinley Bridge the slope varied from 0.17 feet per mile at a 5-foot stage to 0.83 at a 35-foot stage. The intermediate slopes were as given in Table 1. Stages of the rising river were noted from the middle of February, 1908, to the latter part of June. During this time the fluctuations were quite pronounced, and the average rate of rise was about 8 inches in twenty-four hours. Stages of the falling river were noted between the latter part of June, 1908, and the early part of September, when there were no pronounced fluctuations, the general rate of fall being about half the rate of rise.

The substructure for the bridge proper consists of nine piers. The four main piers for the channel spans (Nos. 1 to 4) are of concrete faced with stone masonry. Piers (*A*), (*B*) and (*C*), on the west side and (*D*) and (*E*) on the east side are of concrete. All have caisson foundations, except that the three piers (*A*), (*B*) and (*E*) were founded in open excavation upon wooden piles.

The facing of the main piers (Nos. 1 to 4) is of Bedford limestone, except that the bridge seats and the upstream nose stones above the river bed are of granite. Concrete was used for backing, except for the three courses below the main coping course, which are backed with limestone. The curved surfaces of the upstream starlings are close pointed to ¼-inch projection. The exposed surfaces of

the main copings and the projecting bottom beds of the belting courses are planed. A 4-inch draft line is cut along the lower edges of the belting courses and on each side of the vertical angles of the downstream starlings. All other stones are quarry faced, with projections not exceeding 3 inches.

The design of the channel piers is shown in Figs. 254, 254*a* and *b*. A distinctive feature in the design of the starling coping consists in the manner of closure of the pointed and circular portions. The usual starling coping is dispensed with, and a heavy batter is made from the point of the nose at the base of the starling course to the top of the next course above. This results in a conical surface which was thought to add considerably to the appearance of the pier.

Piers 1 to 4 are 10 feet, and Piers 2 and 3 are 12 feet in width beneath the belting course, with shoulder distances 32 and 35 feet respectively. Piers *A* to *E* are 7 feet wide beneath the coping, which is 30 inches high with a 12-inch projection. These piers are reinforced with steel bars spaced 18 inches apart 6 inches from the surface in shafts and copings.

The piers were located by means of a triangulation system (Fig. 254*c*), the lengths and position of the base lines of which were determined by local conditions, which were far from ideal. The C., B. & Q. Ry. occupied the only available position for a base line on the Missouri shore, and the only advantage of this location was its elevation above high water. The disadvantages were the impossibility of using triangulation houses, interruptions due to passing trains, and heat vibrations over a cinder bed. The length of this base line was 2016 feet, about 66 per cent being to the north of the bridge tangent on account of local conditions.

Collapsible targets were substituted for triangulation houses, each target consisting of a hinged tripod with adjustable target and plumb bob attachment. Hubs with grooved metal tops, in which screws were placed for level adjustment, were driven in the ground for the reception of the tripod tips. This arrangement enabled the target to be accurately set without great loss of time, and was particularly valuable when trains were frequent.

The location of the Illinois base line was made such that the end hubs could be readily seen from the Missouri base line hubs, and local conditions limited its length to 1910 feet. It was placed at as high an elevation as the nature of the shore would permit, and the end hubs were thoroughly referenced on account of the danger of disturbance by flood. Owing to the general unevenness of the shore and intervening gullies, this line was more difficult of accurate measurement than the Missouri base.

Triangulation houses were used on this base line for the protection of the transit and elevation of targets, which were similar to those used for the collapsible targets. The adjustment of the target was made from within the house by means of hook bolts, which extended through posts braced against the roof, the lower ends being in reach of the operator. The target was set on a round base and the bolts were placed at the circumference to allow adjustment at any angle. Two windows, hinged at the top to act as sunshades, were provided on adjoining sides of the house, and the corner posts were made so that a beveled section could be removed from the inside, thus allowing an unobstructed range of vision. The base lines were laid out for measurements at 98-foot points, the tape being supported at quarter points. A 100-foot Chesterman tape standard at 38.1° F. and 12 pounds tension (continuously supported) was used for deter-

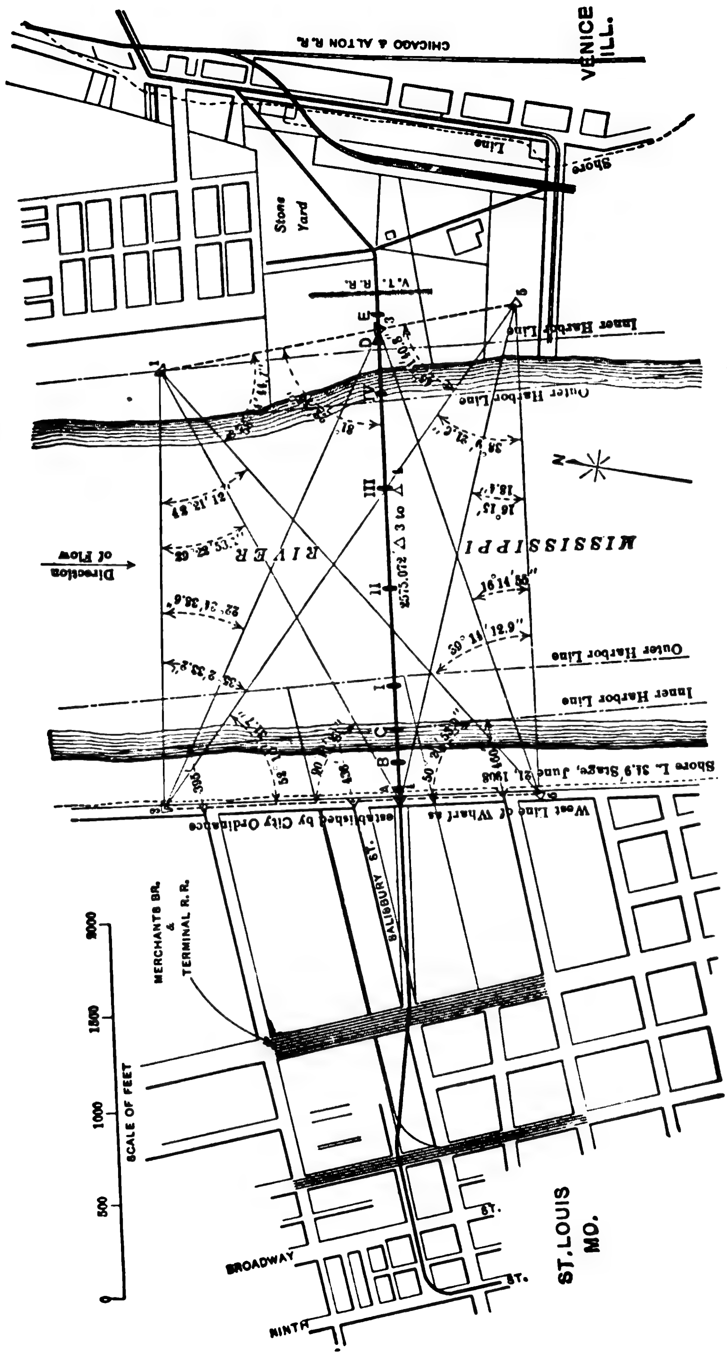


FIG. 254c.—TRIANGULATION FOR LOCATING PIERS OF McKINLEY BRIDGE.

mining lengths. The temperatures were obtained by means of thermometers with exposed bulbs placed on supports at each end and center of the tape.

The observed values were corrected for sag, temperature and difference of elevation. Observations were made during the months of October and November and the range of temperature was from 32 to 83° F. Of 31 measurements, 10 were rejected on account of poor weather conditions and the personal equation of observers. The following results were obtained:

	East.	West.
Maximum variation in observed lengths	0.671	0.0287 ft.
Maximum variation from mean absolute lengths	0.0361	0.0284
Maximum variation in absolute length	0.0626	0.0545
Probable error of the mean	1 in 500,000	1 in 590,000
Probable error of a single observation	1 in 150,000	1 in 185,000

These probable errors denote the relative accuracy in the measurements of the two base lines, but as these errors are a function of the variation of the absolute lengths from the mean, the discrepancies in mean absolute lengths are given as being an index of the probable actual maximum error of the work.

The instrument used for angle observations was a Berger & Sons inverting transit readings to 20 seconds of arc. Each set of observations consisted of at least ten settings with the telescope direct and ten reversed, the repetitions being made in opposite directions to eliminate errors of graduation, of clamp and axis movements, and of adjustment for collimation. The target used was a flat board 8 inches wide and 30 inches high. The face of the target was divided into triangles except for a narrow vertical area through the center which was of sufficient width to subtend 4 seconds of arc for a radius equal to the greatest length of sight in the quadrilateral. This angular width was sufficiently great to extend beyond the vertical cross-hair and allow immediate accurate setting of the instrument. This narrow area and alternate triangles were painted white, the remainder of the surface being red. The white triangles merging into the central white area formed an optical line which was easily located.

The angles of the system varied from 35 to 58°. Small angles were avoided on account of the greater error in resulting distances for an equal error in observed values. The observed values of the angles for the entire quadrilateral were corrected according to the angle equation, and these corrected values were again corrected for the side equation according to the tabular differences for one second for the logarithmic sines of the various angles. The angular values resulting from this method of rigid adjustment were used in all computations for pier location after being verified by computation of base line distances. The co-ordinates of the triangulation stations and piers and the bearings and distances were then computed and tabulated, the bridge tangent being taken as an east and west line for convenience.

St. Louis Municipal Bridge Channel Piers

The channel piers of the Municipal bridge at St. Louis, over the Mississippi, carry the adjacent ends of two 668-feet truss spans, and are correspondingly massive. They are 22 feet 4 inches by 52 feet 4 inches on top, and have a batter of 1 in 24; with a curved end down-

stream below high-water mark, and a cut-water upstream formed by a combination of circular arcs and straight lines as shown (Fig. 255). The piers are of concrete, with rock facing, and having a granite nosing on the cut-water.

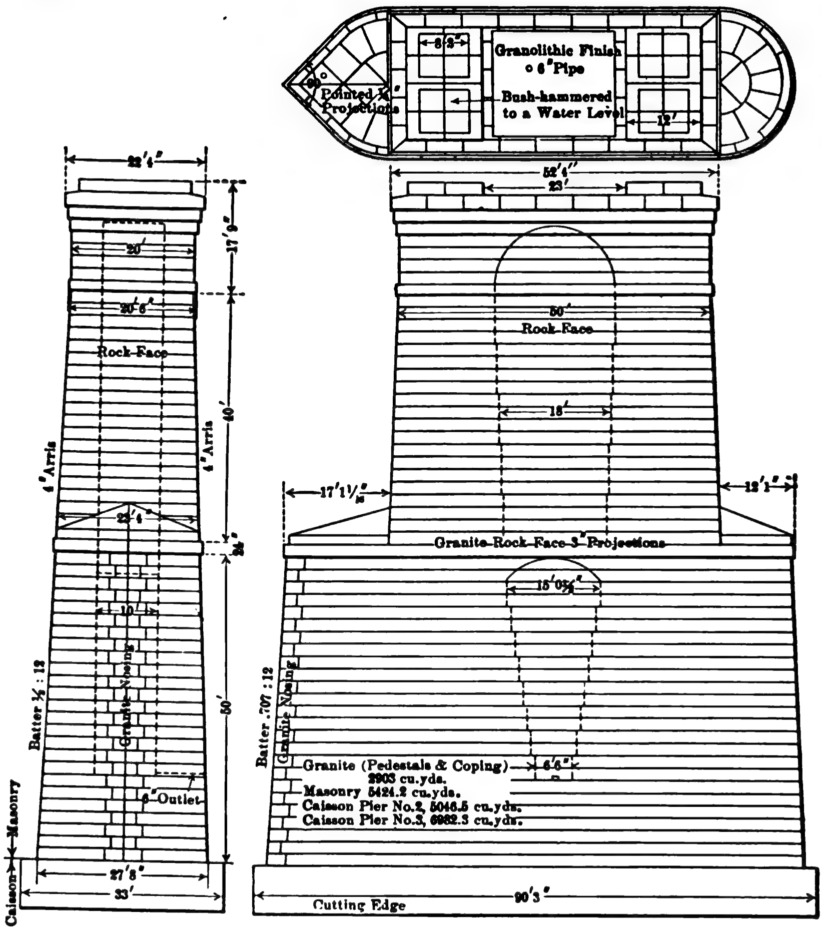


FIG. 255.—CHANNEL PIERS OF THE MUNICIPAL BRIDGE, ST. LOUIS, MO.

The weight and quantity of masonry is reduced by a hollow space above high water, 10 feet by an average of 18 feet as shown, and a smaller hollow space below high water. The appearance of the piers is much enhanced by the double corbel course and by the offset just above high water mark.

The piers were located by triangulating from base lines established on both sides of the river, and the levels were obtained by establishing

bench marks from both ends of the Eads bridge about 1 mile distant. The locations were checked by turning right angles to the Eads bridge, correcting for temperature and grade, and the distances multiplied by the cosine of the angle between the two bridges. The results obtained were surprisingly close to the plan distances, being exactly the same for the west span, 0.03 feet difference for the center span, and 0.12 feet for the east span.

Victoria Jubilee Bridge Ice Breaker Piers

The piers of the Victoria Jubilee bridge over the St. Lawrence river at Montreal, Canada, were built from the plans of Robert Stephenson in 1858, to carry the famous tubular bridge superstructure. They are shown in Figs. 256 and 257, and are undoubtedly the heaviest piers proportionately to the load to be carried, that have ever been built. The reason for this was the fear that it would be difficult to construct any piers that would withstand the ice jams of the St. Lawrence river. The ice breaker has an angle of about 43 degrees and is protected by iron plates. The piers are of great thickness and when the new spans were placed in 1890, it had been found easily possible to increase the length of the piers as shown in the dotted lines, to accommodate the wider bridge. Under the most severe stress from the ice and all possible combinations of loading the masonry is apparently in as good condition as when it was built sixty-one years ago.

These piers may be taken as the superior limit necessary for any possible conditions, and the cut-waters of the Morison type of piers as the lower limit to use for ordinary drift and light ice. The contemplated use of piers even approximating the dimensions used by Stephenson, makes it necessary to give very careful consideration to the obstruction they will cause to stream flow and to the obstruction to the waterway. The author is indebted to F. L. C. Bond, Chief Engineer of the Grand Trunk Railway, and to H. B. Stuart, Bridge Engineer for the data, plans, and photographs of these piers, and for an opportunity to inspect them.

Beaver Bridge Channel Piers

The cantilever bridge over the Ohio river at Beaver, Pa., for the P. & L. E. Ry. had extremely heavy and well-designed channel piers as shown in Figs. 258 and 259.

The final locations of the three river piers varied from their intended

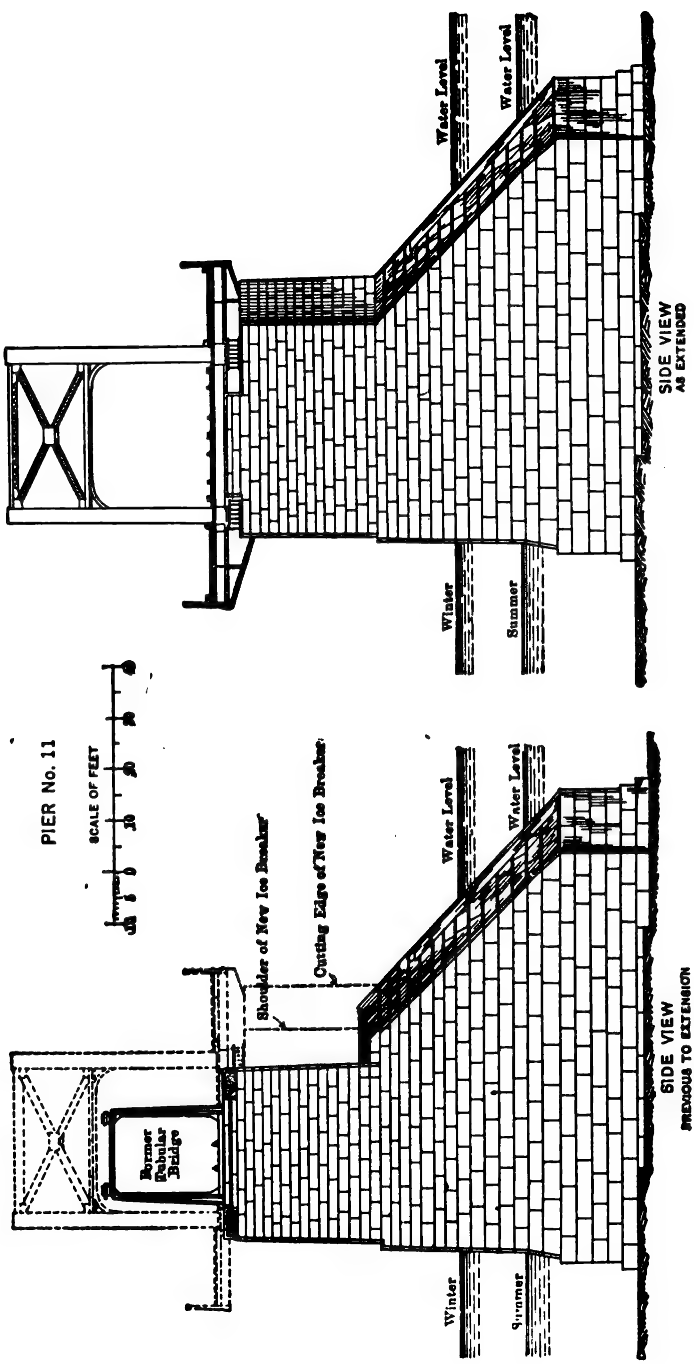


FIG. 256.—PIERS OF VICTORIA JUBILEE BRIDGE, MONTREAL.

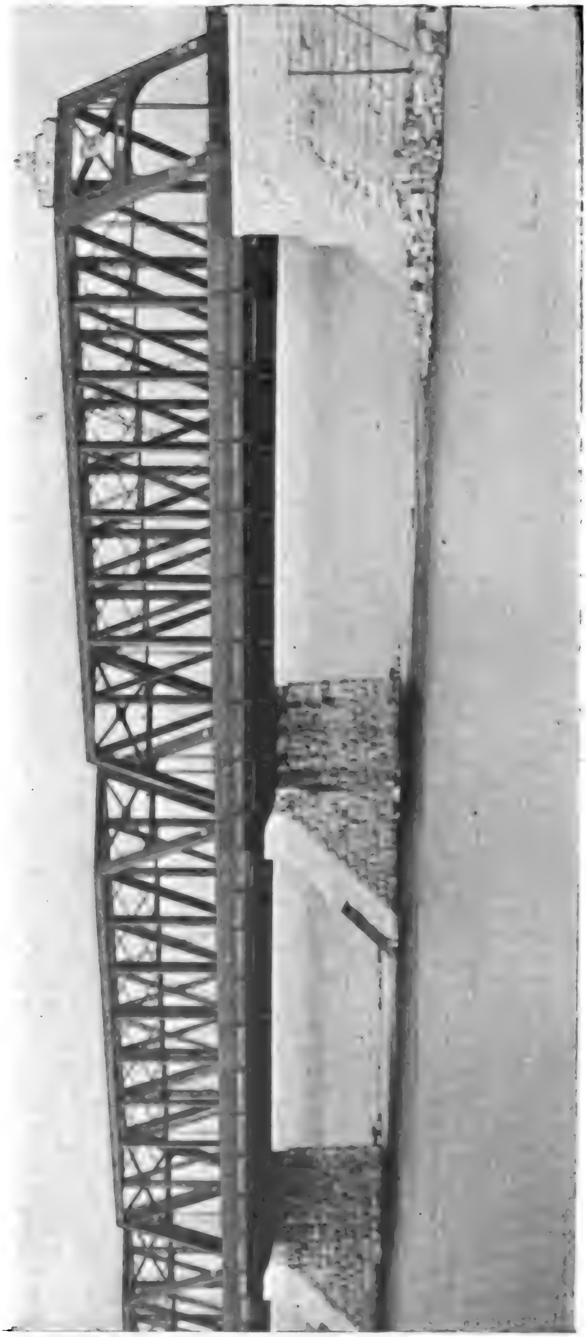


Fig. 257.—Victoria Jubilee Bridge Piers.

location by an average amount of 0.12 feet longitudinal of the bridge and 0.56 feet transverse to the bridge.

The soundings having shown that the rock at the site of the piers was not quite level, the caissons were built with the plane of the bottom edge sloped to fit the rock. However, when the piers were

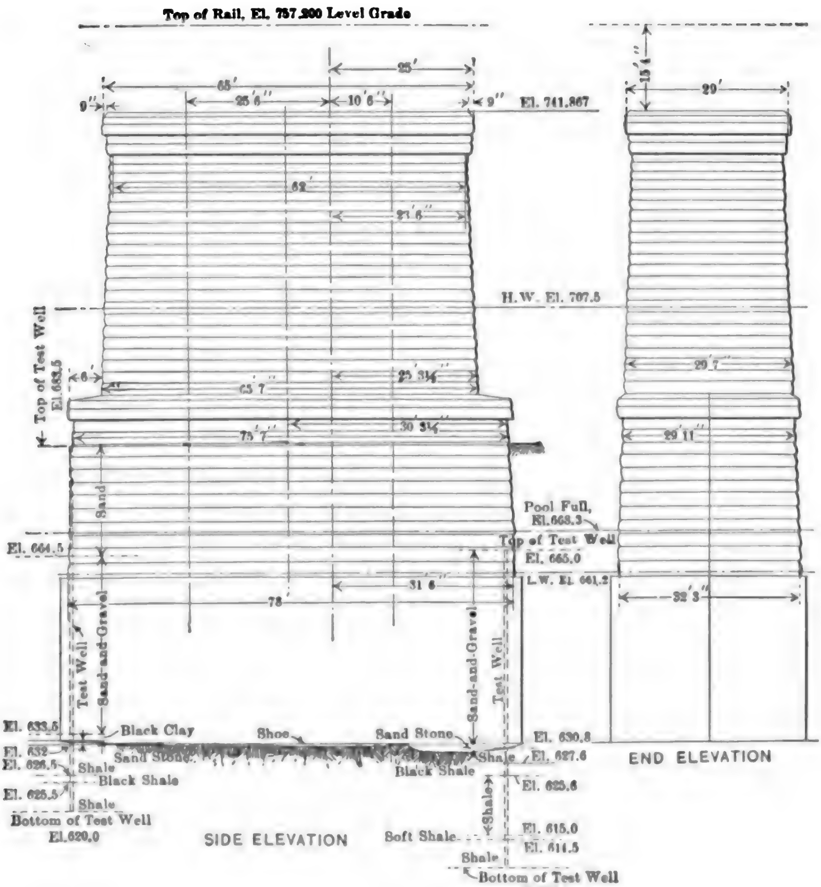


FIG. 258.—SOUTH CHANNEL PIER OF BEAVER BRIDGE.

grounded (cutting-edge about 16 inches above rock), the water flowed under the cutting-edge freely enough to make if difficult even to pump water down to a level 10 or 12 feet above the edge, where forms had to be set for a concrete deck. The walls themselves were watertight, proving the inflow to come under the cutting-edge. The deck completed, air pressure was put on, the remaining material over the rock

excavated, the caissons sunk through the remaining small depth to a bearing on rock, the rock surfaces cleaned off and the whole structure filled with concrete.

The total load on the foundation of each of the channel piers is about 32,000 tons, of which about 12,000 tons is superstructure load.

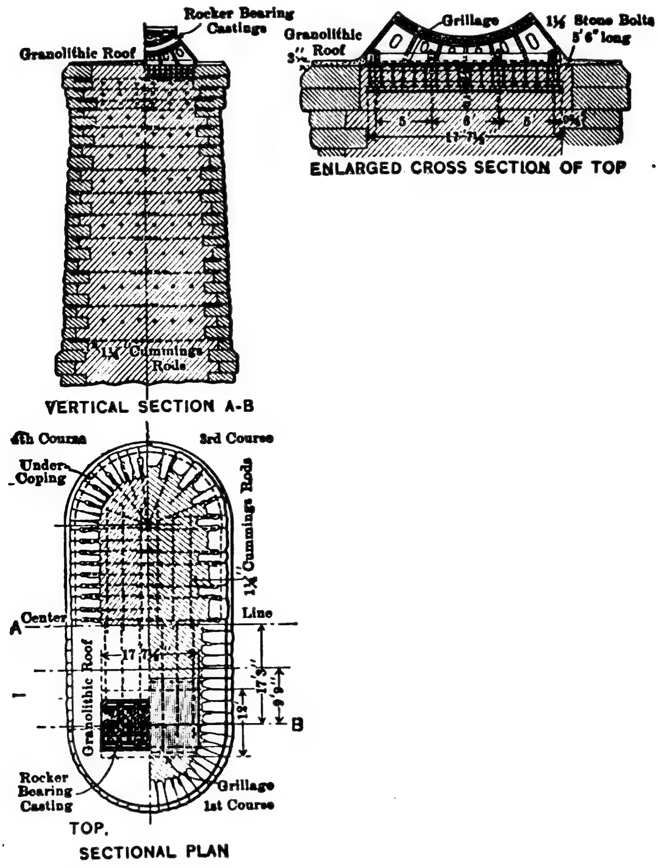


FIG. 259.—SECTIONS OF BEAVER BRIDGE, MAIN PIER.

The pressure per square foot on the rock is about 8 tons from masonry and 5 tons from superstructure, or 13 tons total.

The shoes of the main posts resting on the two channel piers are large steel castings with a circular seat for segmental rollers to form a virtual hinge joint—another of the novel features of the superstructure. The bottom face of the casting is 150 square feet in area, so that the post load of 6000 tons gives a pressure of 40 tons per square

feet. To distribute this over the pier masonry at a more moderate pressure, an I-beam grillage is set under the shoe. This has an area of 240 square feet, making the pressure on the masonry about 25 tons per square foot. In the upper 50 feet of pier below the grillage, $1\frac{1}{4}$-inch rods were placed in alternate layers lengthwise and crosswise in the concrete to assist in distributing the shoe pressure over the full cross-section of the pier.

The concrete forming the body of the piers is a 1.35 gravel mixture, used soft enough to puddle. This applies also to the abutments.

The top surface of the piers is not finished with the customary stone capping. The shoe grillage is set upon and surrounded by 1 : 2 : 4 concrete (as against 1 : 3 : 5 in the body of the piers). The stone outer surfacing of the piers forms only an outer ring on the top face. It was necessary, therefore, to provide a protective surface coat over the pier top. For this purpose a granolithic pavement or roof about 3 inches thick was placed over the entire top surface, out to the inner line of the coping, and carefully finished against the edges of the shoe castings to make a watertight protection. The same kind of top finish was used on the abutments.

The piers tabulated in Tables XLVI and XLVII are 4 feet thick under coping for spans up to 40 feet; 5 feet for spans from 60 to 80 feet; 6 feet for spans from 100 to 125 feet; 7 feet for spans from 150 to 200 feet; and 7 feet 6 inches for spans from 200 to 250 feet. The thickness of footing course has been assumed at 4 feet.

TABLE XLVI.—CONTENTS N. Y. C. RY. TYPE PIERS

Batter on Faces $\frac{3}{4}$ Inch to 1 Foot. Cutwater 4 Inches to 1 Foot. Approximate Contents Cubic Yards for Deck Spans Includes Footings. General Sizes in Note

Height in Feet.	One Track, 12 Feet Long.	Two Tracks, 25 Feet Long.	Three Tracks, 38 Feet Long.	Four Tracks, 50 Feet Long.
8	40	70	100	125
10	50	80	120	150
12	60	90	140	170
14	70	115	160	195
16	80	120	180	225
18	90	135	200	260
20	100	155	230	290
22	110	175	260	325
24	120	200	290	360
26	130	225	320	400
28	150	250	350	440
30	170	280	380	480

TABLE XLVII.—CONTENTS N. Y. C. RY. TYPE PIERS

Batter on Faces ½ Inch to 1 Foot. Cutwater 4 Inches to 1 Foot. Approximate Contents Cubic Yards. For Through Spans. Includes Footings. General Sizes in Foot Notes.

Height in Feet.	One Track, 22 Feet Long.	Two Tracks, 38 Feet Long.	Three Tracks, 54 Feet Long.	Four Tracks, 70 Feet Long.
8	70	110	145	170
10	80	130	165	205
12	90	150	190	240
14	105	170	220	280
16	120	190	250	320
18	135	210	280	360
20	155	230	320	400
22	175	260	360	440
24	200	290	400	480
26	220	320	440	530
28	240	350	480	580
30	260	380	520	630

River Piers of the Knoxville Cantilever

The piers of the Knoxville cantilever bridge, as described in the preceding pages, were five in number, and were built after the plan shown in Fig. 260, except that the length of shaft was 38.2 feet for pier No. 2; 42.5 feet for pier No. 3; 39.5 feet for pier No. 4; 41.4 feet for pier No. 5; and only 19.9 feet for pier No. 6 on the south bank, which had one footing course 2 feet thick and a concrete base 16.7 feet thick on solid rock. The coping in each case was 18 inches thick and contained 16.15 cubic yards; the contents of the piers, including coping and three footing courses was as follows:

	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.
Coping	16.15	16.15	16.15	16.15	16.15
Shaft	448.51	511.53	468.99	496.90	208.23
Footings	118.39	119.63	108.18	116.14	366.72
Total	583.05	647.31	593.32	629.19	591.10

The piers were concrete filled and had a minimum thickness of iron limestone as shown on the drawing, with the inner faces of the stones in each course left rough and projecting into the concrete core varying distances to form a proper bond. The outer faces of the piers

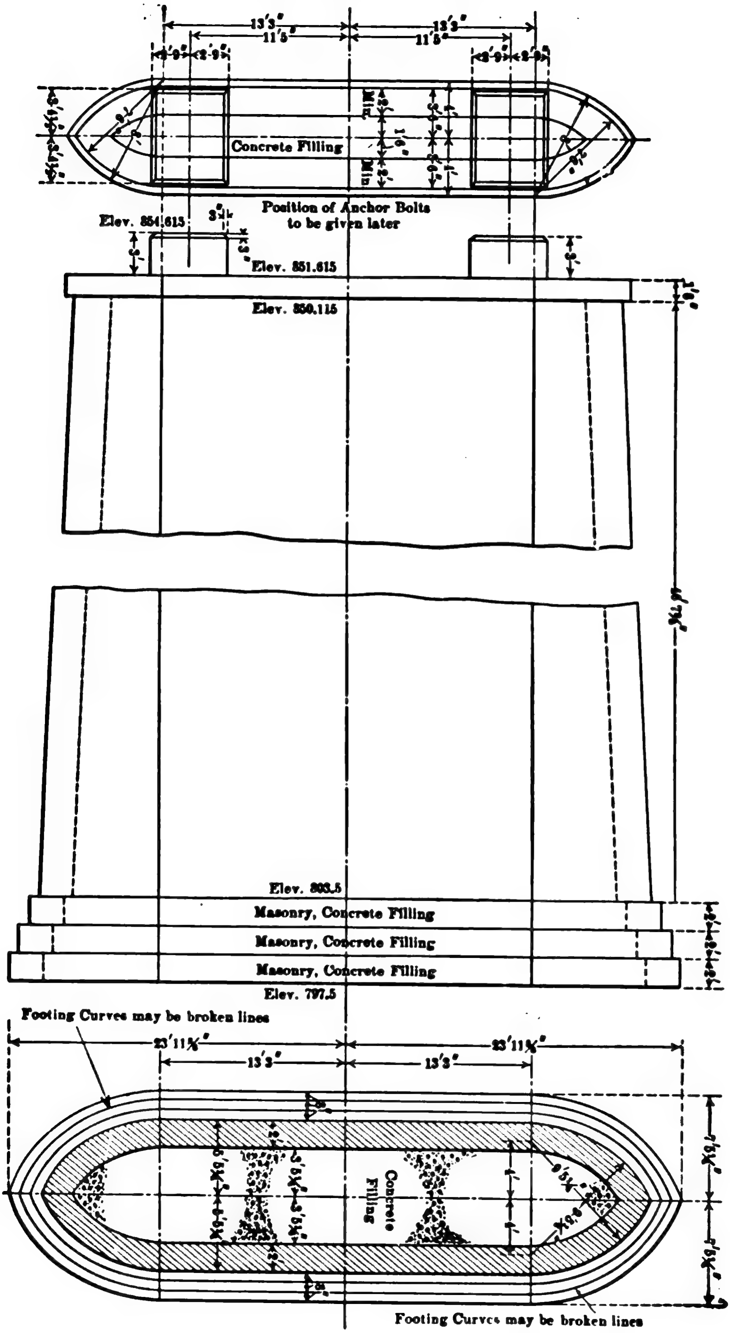


FIG. 260.—PIERS OF KNOXVILLE CANTILEVER BRIDGE.

were rough rock faced, but the appearance of the piers could have been very much improved had money been available for corbel courses and other finish work to enclose the bridge seats. The cross-

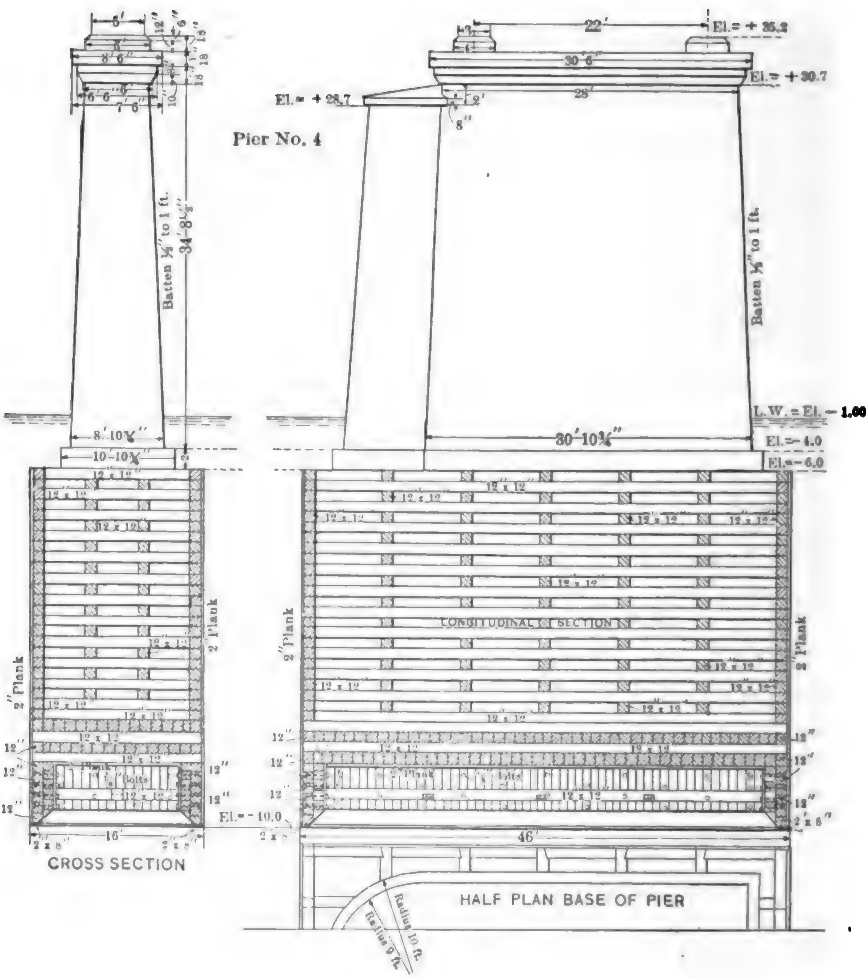


FIG. 261.—CONCRETE PIERS, LITTLE ROCK, ARK.

section of the piers has ends formed of two intersecting arcs of circles, and as the piers extended but a short distance above high water, the same cross-section was used for the entire shaft. The data and plans are taken from the author's record of the construction of the bridge.

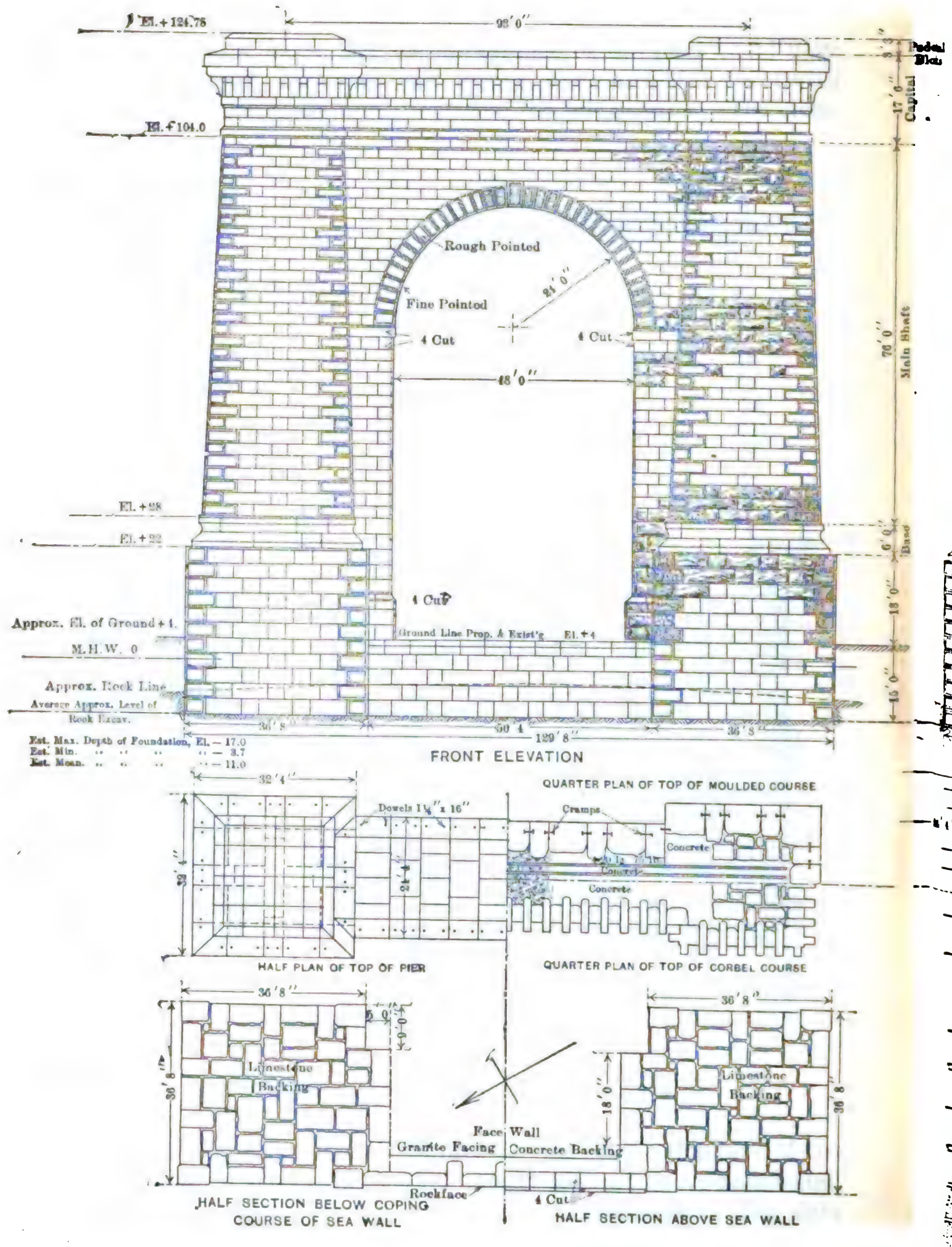
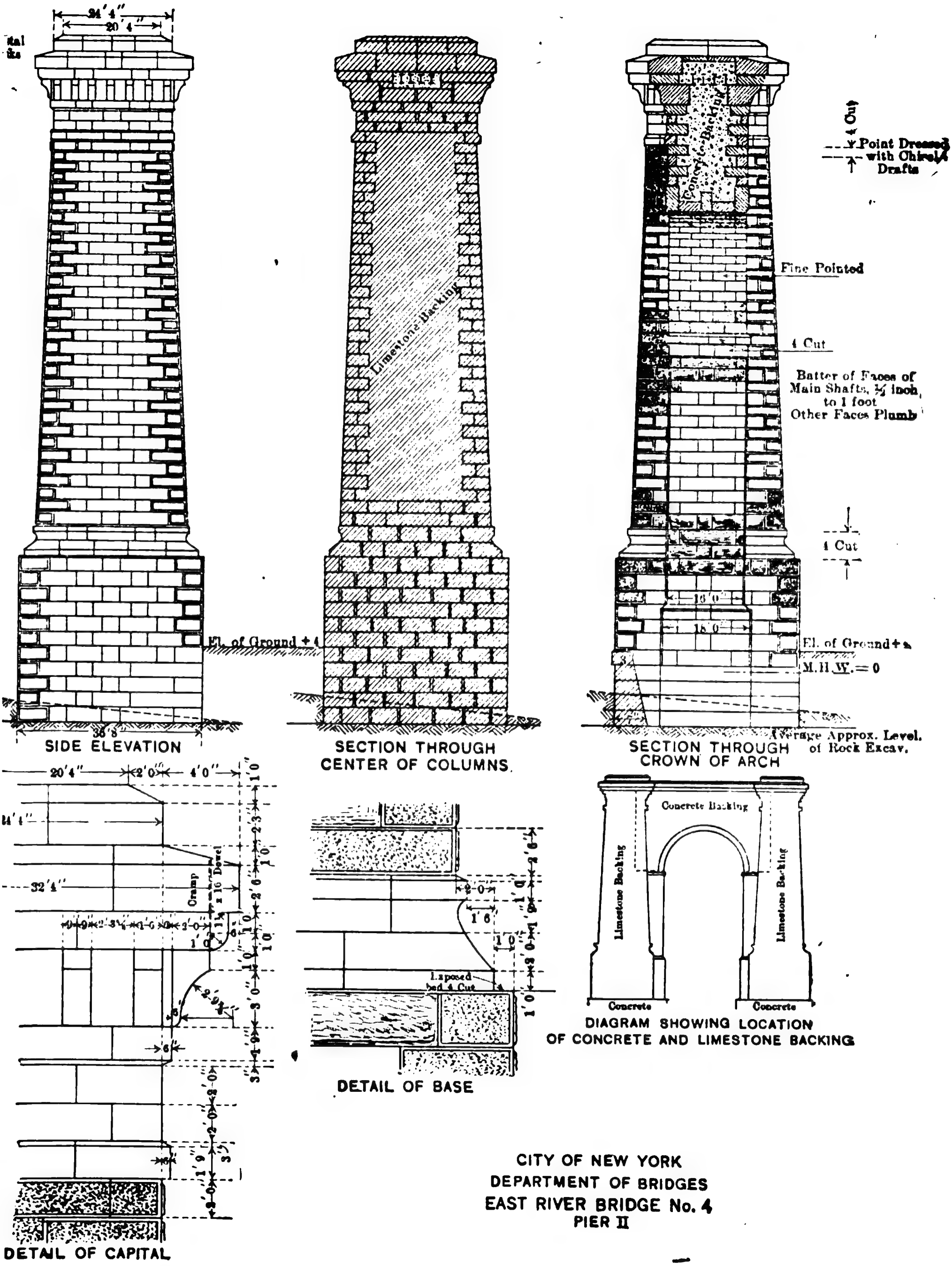


FIG. 262.—BLACKWELL'S ISL



Island Bridge Piers.

[*To face page 427.*]

TABLE XLVIII.—CONTENTS PIERS KNOXVILLE TYPE

Thickness Under Coping 7 Feet. Coping and Footings Fig. 260. Height Shaft Feet. Approximate Contents Cubic Yards.

Height.	Contents, Cu. Yds.	Height.	Contents, Cu. Yds.	Height.	Contents, Cu. Yds.
10	225	25	410	40	615
15	280	30	475	45	685
20	345	35	545	50	755

Plain Concrete Channel Piers

The piers designed by Edwin Thacher, Consulting Engineer, for a bridge at Little Rock were of monolithic concrete and are shown in detail in Fig. 261. The size under coping is 6 feet by 28 feet, with a cut-water formed by two circular arcs of 9 feet radius. The shaft of the particular pier shown is 34 feet 8½ inches in height, with a base course or footing 2 feet in thickness. Particular attention is called to the good appearance due to the corbel course under the coping, which gives a cornice-like effect.

Blackwell's Island Bridge Twin-Arched Piers

The main piers of the Blackwell's Island cantilever bridge are twin-arched piers (Figs. 262 and 263), and are of the very best design and detailing. They consist of two main shafts, connected by an arch resting on pilasters, carefully bonded together. The facing is of rock faced Maine granite, with limestone and concrete backing. The corners of the shafts are quoins, varying uniformly in size from base to capital. The faces of the shafts have a batter of 1 to 24, while the faces of the sub-base, the pilaster, and spandrel walls are vertical. The bases courses proper and the capitals are of four-cut work, and the courses below the capitals are flush with the quoins and are rough pointed, with 2-inch marginal draft.

The masonry in general is ashlar with Flemish bond. The face of the headers is never less than the height of the course, and the length at least two and one-half times the height. The face of the stretchers is not less than twice the height of the course, the width not less than one and one-quarter times the height and they were required to hold their full face thickness for the entire width. The quoins were made to the dimensions shown on the plans, the faces were pointed to

1-inch projections, and all the exposed edges have a 2-inch margin draft. The quoins on the bases and shafts stand out 2 inches beyond the pitch lines of the ashlar.

FIG. 263.—BLACKWELL'S ISLAND PIER.

The ringstones of the arches were set flush, with the arch sheeting and stand out 2 inches beyond the face of the spandrel wall. The soffits of the ring stones were fine pointed and the faces rough pointed, with a 2-inch margin draft all around. Especial attention is called to the careful layout of all the molded courses, which give the piers their finish and add much to their fine appearance.

Hell Gate Bridge, Concrete Approach Piers

The embankment type of construction was not practicable south of 138th Street, because, on account of the greater height, the foundation pressures would have become excessive. The plate-girder type

FIG. 264.—HELLGATE APPROACH PIER.

of viaduct, with concrete piers, was adopted as being most suitable. The spans are generally about 64 feet long, except over the street, where they had to be from 72 to 112 feet. The piers are of plain rectangular shape, with simple square copings.

The general conditions which affected the design of the viaducts in the Bronx section south of 132d Street, on Randall's and Ward's

Islands, and on Long Island north of Lawrence Street, are similar, with the exception of the character of the soil. The fact that these four sections are prominently exposed to view called for a uniform and pleasing appearance, in harmony with the monumental character of the arch bridge which they flank on both sides. The type of viaduct adopted consists of deep plate-girder spans resting on arched concrete piers. (Fig. 264.) The span length varies from 72 to 94 feet, except for a single span over Van Alst Avenue in Long Island, which is 130 feet. The lengths chosen are the most economical, except on the Long Island viaduct, where they were largely determined by the location of the many streets which had to be crossed.

On the Bronx Viaduct the piers rest on cylinder caissons, from 15 to 18 feet in diameter, one under each leg of the pier. These cylinders were sunk partly by open dredging, and partly under air pressure, to depths varying from 45 to 60 feet below the surface, through soft silt and mud to a hard stratum of sand and gravel or solid rock. On all other viaduct sections the piers have ordinary footings built in open excavation. On the Long Island Viaduct, where the soil has various formations of sand, gravel, boulders, loam, and clay, partly saturated with water, which is likely to be drained off, all the footings had to be spread considerably so as to decrease the bearing pressure to the safe value of about $2\frac{1}{2}$ tons per square foot, and had to go to depths as great as 34 feet below the surface.

On Ward's and Randall's Islands the piers have comparatively shallow foundations, either on solid rock, or on hard strata of sand and gravel, or hardpan, which permitted pressures of 4 tons per square foot and greater.

The proximity of the railroad to residential districts and to public parks made it desirable to restrict the noise from trains. Embankments between retaining walls were out of the question on account of the great height. Had soil conditions been favorable throughout a viaduct consisting of a series of solid concrete arches would have been most suitable, as regards appearances and restriction of noise. This type is expensive in first cost, but is more durable and less costly to maintain than a plate-girder viaduct, the steel superstructure of which may have to be replaced or strengthened at some future date, and will require frequent painting. Concrete arches, however, require solid, unyielding foundations, in order to prevent dangerous and unsightly cracks, and such foundations could not be had on every section at reasonable cost.

Fig. 264*a*, *b* and *c* shows typical portions of two preliminary

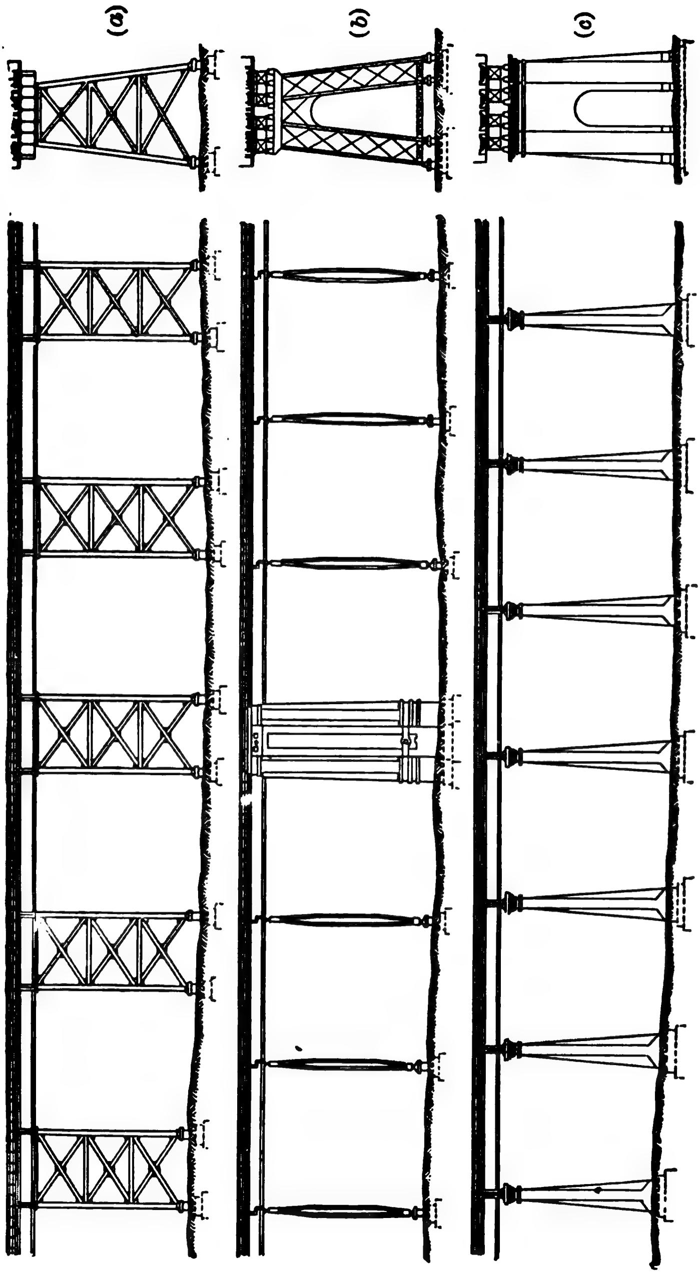


FIG. 264*a*, *b*, *c*.—COMPARISON OF THREE DESIGNS OF PLATE GIRDER VIADUCTS. HELL GATE BRIDGE.

designs for the plate-girder viaducts, and the design adopted. Fig. 264*a* represents the typical American trestle viaduct with alternate tower spans, 40 feet long, and intermediate spans of 80 feet. This type, the advantages of which were cheapness and rapidity of erection, is now gradually being displaced by types of greater rigidity and durability, and better appearance. It would have been inappropriate and inadequate for the approaches to the Hell Gate Bridge. Fig. 264*b* represents the design made by Gustav Lindenthal, Consulting Engineer, in 1906. It consists of plate girders, of nearly uniform span length of from 70 to 80 feet, resting on steel rocker bents. To resist the longitudinal forces from braking and traction solid masonry piers (stability piers) were to be provided at about every tenth span. This design is superior in general appearance to the trestle design. The stability piers convey the impression of rigidity, and give opportunity for architectural treatment. The arch form selected for the steel rocker bents, although somewhat more expensive, is more pleasing than the ordinary two-column bent with single intersection diagonals. This type of viaduct is also stiffer, in the longitudinal direction at least, than the trestle type. Fig. 264*c* represents the type finally adopted, with concrete piers. It is superior to the other two types in appearance, rigidity, and durability, and is less costly to maintain.

A comparison of estimated costs, with the prices prevailing at the time the design was made, showed that, for an average height of viaduct of 100 feet on tangent, the steel trestle design would have been about 20 per cent cheaper, and the design with steel rocker bents about 10 per cent cheaper, than the adopted design. On a 3-degree curve the saving in the first cost would have been only 15 per cent and 5 per cent respectively, as the centrifugal force of the trains requires additional material in the steel bents and towers, but not in the masonry piers. For heights of viaducts of less than 100 feet, the differences in cost are correspondingly less. With the high prices of steel prevailing at present, there would be little, if any, saving in favor of the steel trestle type.

The arched concrete piers mark a radical departure from the ordinary solid square concrete piers with plain surface and simple square coping. The rectangular body of the pier proper is only 6 feet thick, from the coping down, but is reinforced by four buttresses which have a batter of 1 : 15. These piers convey the impression of elegance and yet of great rigidity. The appearance is enhanced by the massive and architecturally elaborate coping of cornices and mouldings. The concrete is made of 1 part Portland cement, 2 parts

sand and 4 parts gravel or broken stone, and is reinforced with steel rods, vertically and horizontally, against shrinkage and temperature cracks.

The account is taken from the paper of O. H. Ammann, in the *Transactions of the American Society of Civil Engineers.*

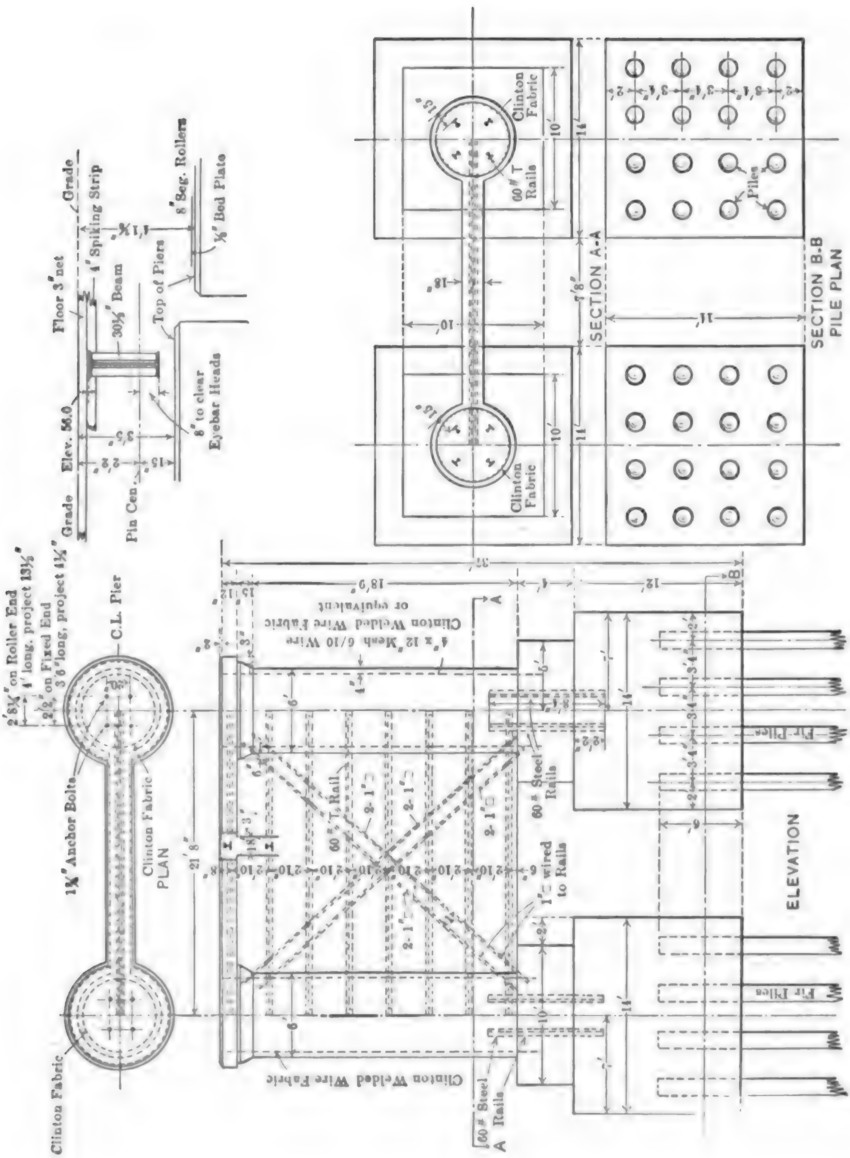


FIG. 265.—REINFORCED CONCRETE PIERS, BELLINGHAM, WASH.

Reinforced Concrete Channel Piers

The piers shown in Fig. 265 were designed for a 380 feet highway bridge at Bellingham, Washington, and are of reinforced concrete of a

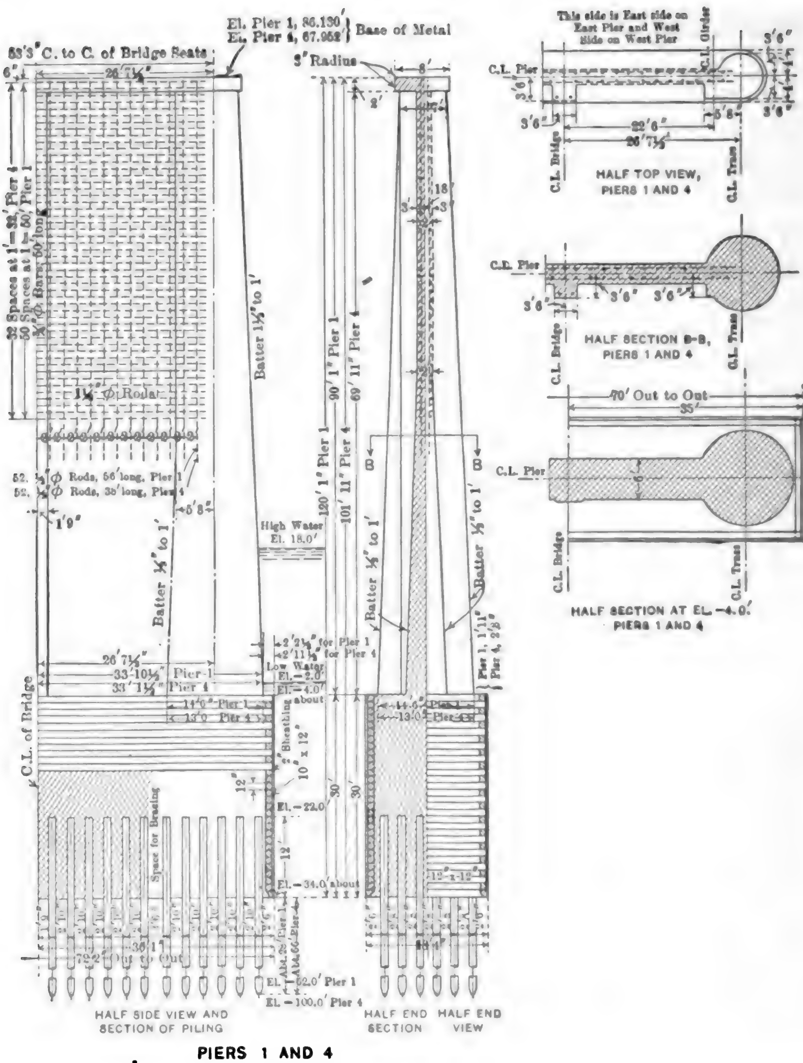


FIG. 266.—TACOMA REINFORCED CONCRETE PIERS.

type patented by the author, but which may be used by anyone without royalty, upon obtaining written permission for each particular case. The foundation for the piers illustrated was of 16 piles

in a crib for each shaft, with a concrete base 14 by 14 feet and 12 feet deep. The footing course is 10 by 10 feet by 4 feet thick which carries the shaft 6 feet in diameter. The shaft is anchored by four 60-pound railway rails, and the same sized rails are placed in the 18-inch webs as reinforcing and to tie the shafts together Diagonal shear reinforcing of the web is also used, and the shafts are reinforced just inside the faces by Clinton electrically welded wire fabric. These piers have a somewhat ornamental top, through the use of the corbel course. Piers of this type when used in a channel, offer somewhat more stream resistance than the best forms of solid masonry piers, but usually this is a negligible matter, unless the spans are short and the number of piers takes up a large amount of the waterway.

The large piers of the Tacoma bridge, as shown in detail in Fig. 266, are of the same type, except the method of reinforcing the webs, and that no reinforcing was used in the shafts. The batter is ½ inch to the foot and the web between the shafts which is 2 feet thick at the top is also battered.

The completed piers are shown in Fig. 107 and the formwork is described in the Chapter on Forms. The concrete was mixed in the proportion of 1 part of Portland cement, 3 parts of sand, and 5 parts of gravel.

TABLE XLIX.—REINFORCED CONCRETE PIERS

Contents Cubic Yards Two Cylinders, Bases and Webs. Fowler Piers, Fig. 265.
Bases Cubical = Diameter + 2 Feet. Webs 1.1 to 1.5 Thick

Total Height Shaft Ft.	3 Ft. Diam., Span 100.	4 Ft. Diam., Span 150.	5 Ft. Diam., Span 200.	6 Ft. Diam., Span 250.	7 Ft. Diam., Span 300.	8 Ft. Diam., Span 350.
10	20	31	45	65	90	119
15	25	39	55	79	108	145
20	30	47	66	93	127	171
25	....	55	76	107	145	197
30	....	63	87	121	163	223
35	....	71	98	135	182	249
40	....	....	109	149	200	275
45	....	....	120	163	218	301
50	....	....	131	177	236	327
55	....	....	....	191	255	353
60	....	....	....	205	273	379
65	....	....	....	219	291	405
70	....	....	....	....	310	431
75	....	....	....	....	328	457
80	....	....	....	....	346	483

Rectangular Reinforced Concrete Piers

The new highway bridge over the Mississippi River for the Citizens Bridge Co., at Burlington, Iowa, has a cantilever channel

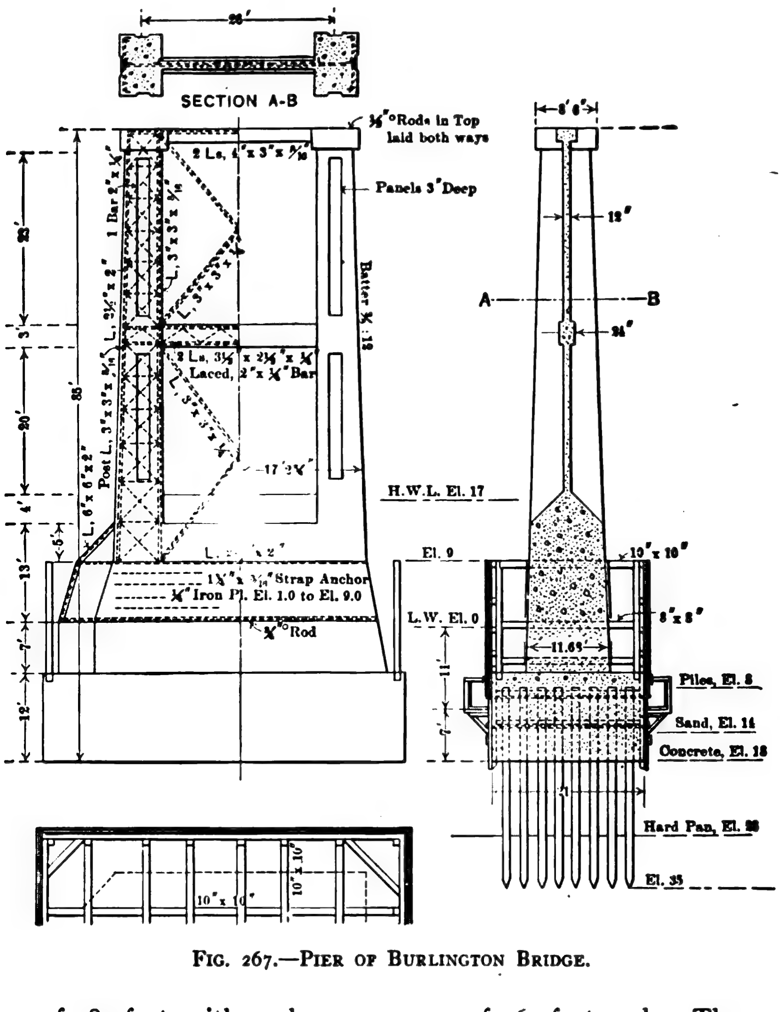


FIG. 267.—PIER OF BURLINGTON BRIDGE.

span of 480 feet, with anchor-arm spans of 260 feet each. There are five concrete piers. Three of them have pile foundations, while the others are founded on rock. The general design of the two channel piers is shown in Fig. 267. Each consists of a pair of reinforced-concrete pedestals, paneled on three sides and connected by a 12-inch curtain wall, or diaphragm. The reinforcement is of structural steel.

All the concrete below water in the tubular piers, and in the main piers to within 6 feet of low water mark, was run through 10-inch tremies of No. 16 galvanized iron. These were 15 to 30 feet long. When first used, they were suspended from the boom of the hoisting tower by a line running direct to the drum of the hoisting engine. Later it was found that this direct connection was unsatisfactory; and a pair of double blocks was put in, in order to keep better control of the elevation of the tremie, which had to be gently

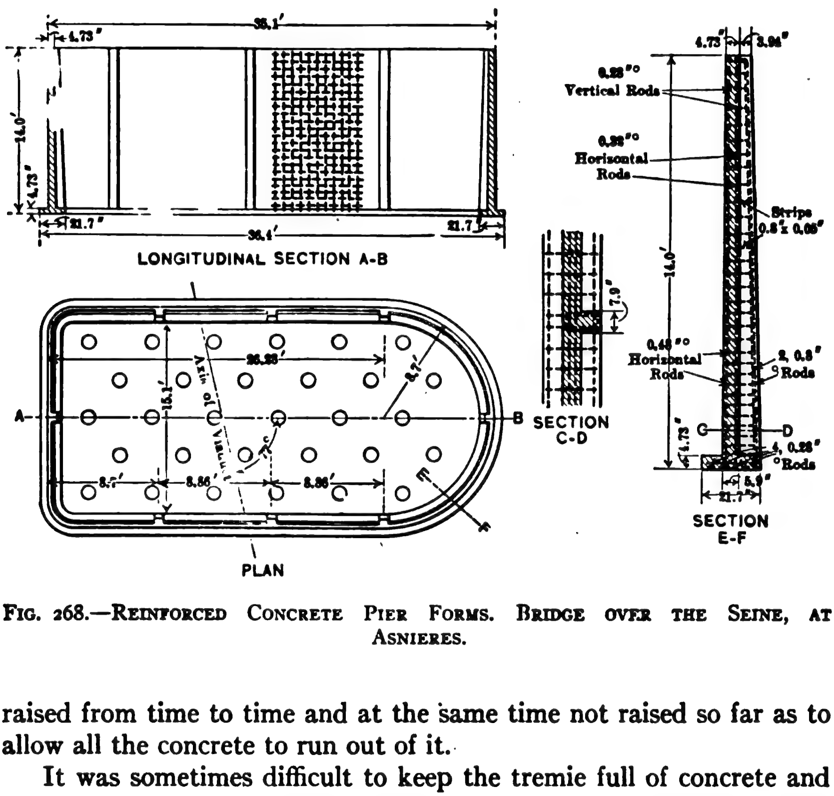


FIG. 268.—REINFORCED CONCRETE PIER FORMS. BRIDGE OVER THE SEINE, AT ASNIERES.

raised from time to time and at the same time not raised so far as to allow all the concrete to run out of it.

It was sometimes difficult to keep the tremie full of concrete and not run over, but with the exercise of care and by tapping the side of the tremie this difficulty was soon overcome. The concrete under water was a 1 : 2 : 4 mixture, that above water a 1 : $2\frac{1}{2}$: 5. The forms were of 1-inch lumber, with $3\frac{1}{2}$ by 6 inch studding tied across with No. 10 annealed wire, which was kept tight by using eyebolts. These bolts were placed between two $3\frac{1}{2}$ by 6 inch wales spaced about 5 feet center to center. As piers Nos. 3 and 4, and Nos. 2 and 5 were alike, the forms were used over again.

The above account is taken from *Engineering News*, March 8, 1917.

Pier Forms of Reinforced Concrete

The forms for piers when there are a large number of duplicates are made in sections to be used over several times, even if of timber. The use of metal forms makes it possible in such a case, to use them over again throughout the work. There are some cases where it would be more convenient or cheaper to make them of reinforced concrete, as was done for the piers of the new bridge over the river Seine at Asnieres, France.

The piers were 15 feet thick by about 32 feet long, and 14 feet high. The forms were 4.73 inches thick, with 3.94 inch ribs as shown in Fig. 268. The horizontal rods were practically $\frac{1}{2}$ inch in diameter, spaced about 5 inches apart, and vertical rods of about $\frac{5}{16}$ inch diameter. The ribs were reinforced by $\frac{7}{8}$-inch rods, and the thickness of the ribs was 7.9 inches.

Hollow Piers for Forth Bridge Approach

The end approach piers of the cantilever bridge over the Firth of Forth in Scotland were built hollow as shown in Fig. 269 to save unnecessary weight on the foundation bed, and also to save in masonry cost. The height of the one illustrated, or the south end pier is 189 feet 6 inches to the rail level, and 243 feet 6 inches to the top. The foundation is carried down to a bed of boulder clay 33 feet 6 inches below high water, and a very heavy coffer-dam was required, 76 feet by 127 feet in plan. This was divided in half by a double row of piles with puddle between, and when the concrete was part way up, this center dam was cut off and the concrete carried solid across. The south end pier rests on the whinstone 21 feet above high water.

The piers have a general batter of about 1 to 30 and a curved batter at the lower part of the shaft, which gives a fine architectural appearance. The base has rounded ends, but the shaft is square ended. The hollows in the base are five in number and are arched over at the top, but the hollows in the shaft are flat topped as shown in the sections. The facing is of Aberdeen gray granite, and the backing is of concrete in the proportions of 5.5 cubic feet of Portland cement, 5.5 cubic feet of sand and 27 cubic feet of broken whinstone.

The same idea of using hollow piers has been used in reinforced concrete, with an exterior reinforced shell and tie walls of the same type. The effect is of a greater solidity than can be obtained by using piers of the type shown in Figs. 265, 266 and 267, but the hollow type will usually require more masonry and the spaces are likely to accumulate dirt and hold water, unless large drain holes are provided and kept clean.

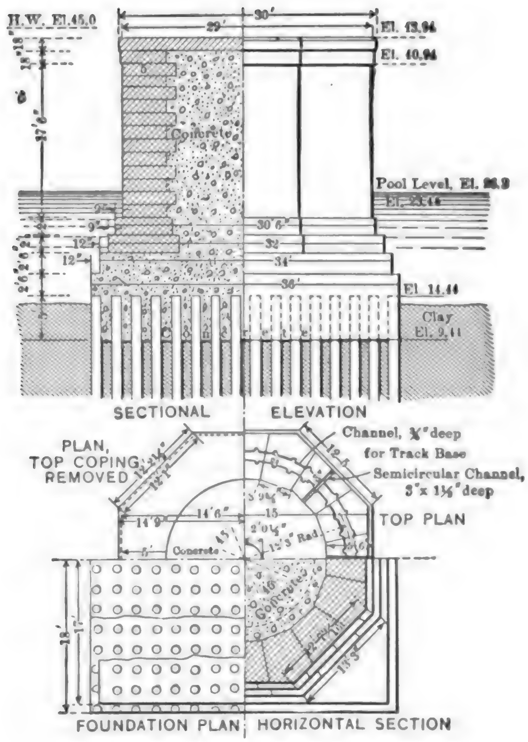


FIG. 270.—PIVOT PIER, ILLINOIS RIVER BRIDGE.

Circular or Pivot Piers for Swing Spans

The design of pivot or circular piers for swing spans is comparatively simple, as there is usually a small pressure per square inch to be provided for under the track and the center bearing. The weight of the pier itself is the most serious feature and hollow masonry shells with tie walls are usually desirable. The pier for the Ft. Madison bridge of the Santa Fe Railway is shown in Fig. 19, page 34,

and is concrete filled and for center bearing. The pier for the Harlem Ship Canal swing bridge is shown in Fig. 29, page 50, and is also concrete filled with a concrete base. The pivot pier of the Chelsea bridge at Boston is a concrete pier with stone facing, and is shown in Fig. 31, page 52.

The pivot pier of the Illinois river bridge for the Peoria and Pekin Terminal Railway is shown in full detail in Fig. 270. The shaft is octagonal 29 feet in diameter and 17 feet 6 inches high. The corbel course is 29 feet 6 inches in diameter by 18 inches thick, while the coping is 30 feet in diameter and 18 inches thick. There are two 9-inch offsets in the masonry and two 12-inch offsets in the concrete. The base is of concrete 36 feet square, with 5 feet of con-

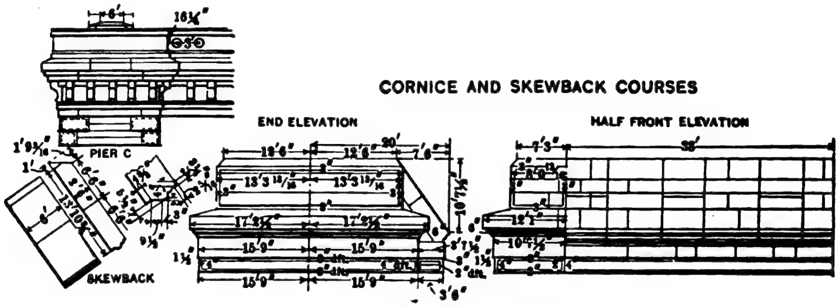


FIG. 272.—DETAILS WASHINGTON BRIDGE PIERS.

crete over the tops of the 256 piles, which are driven into the clay bed of the river under from 12 to 14 feet of water. The pier has a masonry shell of 5 feet, and 6 feet alternate courses filled with concrete drainage channels 3 inches by 1½ inches are provided under the track.

Skewback Piers of the Washington Bridge

The Washington bridge over the Harlem river in New York city is one of the finest pieces of bridge architecture in the world, and the masonry is particularly notable for its solid construction, and the perfection of its design and detailing. The center pier (Fig. 271) is especially notable, as it carries the adjacent ends of the two 510-feet two-hinged steel-arch spans. The masonry is of concrete faced with Maine granite in from 2 to 3 feet courses, having as much bed as rise and laid with ½-inch joints. The quoins are rock faced from 3 to 6 feet on the front, and have a 2-inch flat draft around the face. The shaft of the pier is 25 by 90 feet, and the base is 36.55 by 100.16

feet. The skewbacks are only net size 4 feet 4 inches by 13 feet and the appearance would have been much improved by making them with considerably more margin around the steel shoes.

The details of the skewback courses and of the cornice shown in Fig. 272 are especially worthy of study and emulation in the designing of new work. They are of cut stone and add very largely to the general appearance of the masonry and of the whole bridge. The entire structure was designed and detailed under W. R. Hutton, Chief Engineer of the Bridge Commission.

APPENDICES

SELECTIONS FROM SPECIFICATIONS

APPENDIX I

SPECIFICATIONS FOR COFFER-DAMS AND FOUNDATIONS, OHIO RIVER MOVABLE DAMS

MAJOR W. H. HEUER, U. S. Engineer

GENERAL DESCRIPTION

The site of Dam No. 2 is on the Ohio River, distant from Pittsburgh, Pa., 10½ miles, and adjacent on the right bank to the Pittsburgh, Ft. Wayne and Chicago Railway. It has Neville Island on the left bank, and is accessible by street cars from Pittsburgh.

The lock is to be located on the left bank of the Ohio River, immediately behind Merriman's dyke. It will be in general dimensions the same as locks Nos. 1 and 6, viz., 110 feet wide and 600 feet long.

SPECIAL DESCRIPTIONS

The river bed at No. 2 consists of gravel throughout, and the excavations will be made to a depth sufficient to insure a permanent and enduring foundation, which will ordinarily be 14 feet below the gate sill, but may be otherwise, as the engineer, in his judgment, may direct.

The work will conform to the drawings exhibited, and to such others, in explanation of details or modifications of plans, as may be furnished from time to time during construction.

CONTRACTOR TO FURNISH ALL MATERIAL AND WORK.—It is understood and agreed that the contractor, under his contract prices for work in place, is to furnish and pay for all materials, stone, cement, sand, earth, timber, material for coffer-dam and protection cribs, excavation, lock-filling and discharging valves (set in masonry), flushing valves, anchor bolts, lock-gate tracks, and everything entering into or connected with either the permanent or temporary construction, and he is also to supply and pay for all work, skilled and other-

wise, required to prepare and place the materials, and complete the work according to the drawings and these specifications.

CONTRACT TO INCLUDE.—The contract will cover the construction and completion of the foundation, masonry and timber work of the lock, including both land and river walls, the gate-recess walls, the foundations of the lock-gate tracks, the guiding walls above and below the lock, the pipe and flushing conduits, the drift chute, the foundations for the power-house and lock-keepers' residence, and every such other permanent construction as shall be shown upon the drawings. It shall also include the clearing of the land necessary for the proper execution of the work embraced in this contract, all pumping and bailing, dredging and excavation, puddling and embankment, the construction of all coffer-dams, stone masonry, concrete and brick masonry, timber work and iron work, and all such other work which, in the judgment of the Engineer, is necessary and included in the proper completion of the contract.

TOOLS, MACHINERY, BUILDINGS, ETC.—The contractor, without cost to the United States, shall furnish all appliances, dredges, pumps and pumping machinery, boats, tools, derricks, tramways, foot-walks, roads and landings, and all needful temporary buildings and shops.

COFFER-DAMS

SHEETING.—The coffer-dam, about 1500 feet in length, shall be built as shown generally by the drawings exhibited, and as directed by the Engineer. It shall be 14 feet high above the sill of the lock, and shall consist of two walls or rows of plank sheeting, spaced 12 feet apart in the clear, driven or set firmly from 1 to 2 feet into the river-bed, and supported laterally by horizontal longitudinal stringers, the latter being spaced at varying intervals, increasing in width from the bottom to the top, and to be sufficiently and firmly bolted together transversely with iron rods passing through the coffer-dam horizontally from the rows of stringers on the one side to the corresponding rows on the other, against which the vertical plank sheeting shall be securely spiked.

FILLING AND DECKING.—The interior, or space between the walls of sheeting, shall be filled with heavy dredged river-bed or other material not liable to wash, and to be covered over with a suitable decking of plank (to protect it from injury in case of being submerged by floods), all complete as shown on the drawings.

PILING AND CRIBS TO PROTECT.—At the upper outer corner of the coffer-dam shall be placed a crib built of framing timber and filled with riprap stone; from the upper corner of the crib, at an angle of 45° with the axis of the current, a line of piling, spaced 5 feet apart, firmly bolted together with waling-pieces, shall be driven to the shore to form a protection to the coffer-dam; also outside and along the coffer-dam, from the upper outer corner to the lower corner, clusters of piles, firmly bound or bolted together, shall be driven at intervals of about 80 feet. The tops of all piling shall be sawed off to a uniform height of 2 feet above the coffer-dam. Protection cribs shall be placed at such other points along the coffer-dam as may be shown upon drawings.

HOW PAID FOR.—Bidders will state a price per lineal foot of coffer-dam completed. No payment will be made for any portion thereof until the entire coffer-dam is conpleted. Drawings will be furnished, showing the general type of the coffer-dam and its manner of construction, and every detail necessary for intelli-

gent bidding. Should any work on the outside of the coffer be necessary, such as gravel filling or riprapping, it shall be paid for at the price bid for gravel filling, riprapping, etc. If, owing to the nature of the river bed, it shall be found impossible to drive the plank sheeting to the required depth, then the contractor, after driving the sheeting as deep as possible without injury, and in lieu of driving it to its full depth, shall fill around the outside of the walls with the same material as is used in filling the coffer-dam, to the height of 4 feet above the surface of the river-bed, and for which no extra compensation will be allowed.

Removal of.—The contractor will be required to remove the coffer-dam and its belongings at his own cost. The time and manner of the removal of the coffer-dam, or any part thereof, and the place to deposit the materials shall be prescribed by the Engineer.

To Belong to the United States.—It is understood and agreed that the payments made for the coffer-dam, including the crib and pile protection, shall cover the entire cost thereof to the United States, and by virtue thereof they shall become the property of the United States. The contractor, however, must maintain the same and make all needed repairs to same during the existence of the contract, without expense to the United States.

Deposit within the Coffer-dam.—Material washed or left in the space inclosed by the coffer-dam by freshets shall be removed by the contractor, as directed, at his price for common excavation, which price shall cover all necessary cleaning and scrubbing. No payment will be made, however, for removing material washed into the inclosure from the coffer-dam itself or from any deposit made by the contractor on or above the works.

MATERIAL AND WORKMANSHIP

Temporary Piling shall include all piles driven for the protection of the coffer-dam and " deadmen " for derricks. They shall be of good quality, round oak timber, not less than 12 inches diameter at the butt, and of length varying from 20 to 25 feet, and longer if necessary.

Sheet Piling.—In excavating for foundations, should quicksand or fine sand carrying water be encountered, close sheet-piling will be required to be driven to whatever extent the Engineer may direct.

Sheeting.—The sheeting shall include the walls and decking of the coffer-dam, including the stringers; also such shoring as may be directed by the Engineer to remain in the finished structure. It shall consist of the best quality of hemlock obtainable, and must be in all cases satisfactory to the Engineer in charge.

Gravel or Earth Filling.—Gravel or earth filling will include all material used in filling the land-wall inclosure, back of the guiding walls, etc. It does not include any filling in the construction of the coffer-dam.

Stone Filling shall include all stone placed in the protection cribs or any riprap stone ordered for the protection of the work.

Cribwork shall be built of hemlock framing timber framed together in square bins and securely bolted together by iron drift-bolts. The interior of the cribs shall be filled with riprap stone, and should the Engineer deem it necessary such riprap stone shall be placed on the outside of the crib. The whole to be built as shown by the drawings.

Framing Timber.—For all temporary cribwork, also the permanent crib at the head of the upper guiding wall, framing timber shall be used. No stick shall be less than 10″×10″ in section.

" Framing timber " is a commercial term for a class of timber hewn to various sizes.

EXCAVATION

To Include.—It shall include the removal of all gravel or other material to the depth required for the lock and its upper and lower entrances, the gate recesses, Poiree-dam and gate-track foundations, for the foundations of all walls, and for all conduits or wells, and all such other material as may be found necessary in the judgment of the Engineer to be removed for foundations and otherwise in permanent construction. It will include all dredging and all material excavated of whatever nature, however removed, for foundations and for site of coffer-dam.

Lines, Slopes, and Grades for.—All excavations shall conform to such lines, slopes, and grades as may be given by the Engineer, and anything taken out beyond such given limits will not be paid for by the United States.

Material to be Deposited.—Excavated material is to be deposited as and where directed by the Engineer. It shall be deposited in such manner as not to interfere with present or proposed navigation. Material of any kind deposited by the contractor in absence of, or in disregard of, instructions, shall, if required by the Engineer, be removed by the contractor at his own cost.

Shoring.—All excavation for foundation shall be securely shored and thus maintained until the foundation has been sufficiently advanced to dispense with the same, when it may remain or be removed at the discretion of the Engineer.

Dredges and Pumps.—The contractor will be required to employ, at the same time, not less than two suitable steam dredges at excavating and filling; and for pumping he must keep at least three good sufficient pumping outfits, with pumps, engines, and boats complete, in or always ready for operation. The dredges must be equipped to do effective work to a depth of 28 feet.

FOUNDATIONS

Changes or Modifications of.—The character of the river-bed and of the proposed foundations for the different parts of the work is shown in general on the drawings and cross-sections exhibited, and it is understood that the United States shall have the power to make any changes in the plans of the foundations that may, in the judgment of the Engineer, be considered advisable after examinations made, as the excavations proceed within the coffer-dam after it is pumped out, and it is understood and agreed that the contractor shall have or make no claim against the United States on account of any such changes in or modifications of the plans of the foundations, or on account of any increase or decrease in the depth of same, under or over those referred to herein or shown on the drawings exhibited.

MASONRY

Cement.—Cement will be of uniform quality, setting well both in air and water, and free from anything that will cause the mortar to swell, crack, or scale. It shall be put up in strong, sound barrels, well lined with paper so as to be reasonably protected from air and moisture: The average net weight of the barrels shall be not less than 265 pounds, unless expressly so stated in the proposal. Each barrel must be labeled with the name of the brand and of the manufacturer.

In general, ten barrels of every one hundred will be tested.

The cement must stand the following tests: Fineness—At least 85 per cent must pass a sieve of 6400 meshes to the inch. Setting—Cement must be moderately slow setting; it must not begin to set within fifteen minutes, as determined by Vicat needle $\frac{1}{12}$ inch in diameter with $\frac{1}{4}$ pound load, and it shall not bear weight of one pound on wire $\frac{1}{24}$ inch in diameter within thirty minutes, but must bear such weight within one hour and a half. Strength—The minimum tensile strength per square inch of briquettes of neat cement mixed with about 33 per cent of water by weight, and exposed in air for one hour, and the remainder of 24 hours in water, shall be not less than 50 pounds; with longer time, whether in air or water, there must be a decided increase of strength; it must also test to the satisfaction of the Engineer when mixed with sand. The tests for setting will be made at a temperature of air and water of about 75° Fahrenheit. All other tests will be made at a temperature above 60° Fahrenheit. The cement will be subject to inspection at all times, and must be kept well housed.

Sand.—The sand used must be clean, sharp, washed, river sand, satisfactory to the Engineer.

Mortar.—To be composed generally of two parts of sand to one of cement; when required, and whenever thought necessary by the Engineer, it shall be made richer. It must be thoroughly mixed and used before it has begun to set. If required by the Engineer, the mortar beds will be protected from the sun.

Pointing.—All face work is to be pointed, as fast as the work progresses, with stiff mortar, mixed one of sand to one of Portland cement, thoroughly hammered in and finished with proper tools; before the final acceptance of the work all face masonry which at that time does not appear properly pointed shall be repointed by the contractor to the satisfaction of the Engineer, without extra cost.

Frost.—Masonry will not be executed during freezing weather, nor when, in the judgment of the Engineer or his agent, it is likely to freeze before the mortar shall set. To guard against injury from frost, all new and unfinished work shall be properly protected by the contractor at his own cost.

Voids and Openings.—Due regard shall be had in the construction of all masonry walls to leave all necessary voids or openings for conduits or wells, or for such other purposes as may be required by the Engineer.

Ashlar.—It shall comprise such part of the walls as is built of stone, with point-dressed face, and beds, and joints smoothly and squarely dressed.

Quality of Stone.—All stone shall be perfectly sound, strong, hard, free from injurious seams, and in all respects satisfactory to the Engineer. Stone to be such as can be truly wrought to such lines and surfaces, whether curved or plain, as may be required. No stone shall be used which weighs less than 135 pounds to the cubic foot.

Samples of Stones. Each bidder must deposit at this office, all charges prepaid, before the bids are opened, a 6-inch cubical block of the stone he proposes to furnish, and state the quarry from which it was obtained. The quality of the stone must be at least equal to that of the sample. The sample must be truly squared, and dressed on four sides; one side must be hammer-dressed, one side smooth-dressed and rubbed, and one side pitch-dressed. The other side is to be left with quarry face.

Stone May be Rejected.—The United Stated reserves the right to reject any stone not deemed suitable, or which, during the execution of the contract, shall be found defective. The beds of the stone must be their natural quarry beds. No lewis or dog holes, letters, or marks of any kind will be allowed on any dressed face of stone, but each face shall have left on it a boss for lifting, to be removed by the contractor after the stone has been set.

Dressing of Stone.—Stone must be accurately cut, square, and true, and the faces must be pitch-draughted and point-dressed to a plane with the draught, forming an approximately smooth surface. The beds must be smoothly and squarely dressed, full length and width. The vertical joints must be dressed to a depth of not less than 18 inches from the face, and the allowance for joints must not exceed $\frac{3}{8}$ inch. One-third of the stone in each course must be headers. All stones not accurately dressed will be rejected. All dressed stone must have the dimensions plainly marked on one end.

Dimensions.—The cut-stone stretches must be not less than 3 feet nor more than 5 feet long, and their width must not be less than $1\frac{1}{2}$ times the height of the course to which they belong. The width of the headers must be not less than $1\frac{1}{2}$ times their height, and their length must be at least double their breadth, unless otherwise ordered. The thickness of course includes the joint, which will be $\frac{3}{8}$ inch.

Laying Stone Masonry.—The faces of the wall shall be accurately laid to the lines indicated on the drawings, or as directed by the Engineer. All stones to be well laid to proper lines, in full beds of mortar, and settled in place with a wooden maul; the use of grout is prohibited. No dressing, except in special cases, and by permission of the Engineer, will be allowed on backing after it is laid in the wall. The bond of stone shall in no case be less than 9 inches. The walls will be laid in horizontal courses throughout, each course to be of uniform height through the wall. Heights and arrangements of courses to be determined by the Engineer. When laying masonry the site for the stone shall be thoroughly cleaned with a scrub-broom and moistened; and the stone shall always be cleaned and well moistened before being set. Not more than three unfinished courses of face stone will be permitted upon the wall at the same time without special permission from the Engineer in each case. Proper machinery must be used in handling the stone; face stone shall not be disfigured by use of plug or grabs. Any stone chipped or spalled shall be rejected. Stones having defects concealed by cement or otherwise will be rejected on that account alone.

Coping.—The coping will be of the same class and quality of stone described in ashlar masonry. It will be carefully and truly cut to forms and dimensions given, from the best stone; it will be crandalled on all outer faces; the exposed edges of the coping to be rounded to a radius of 3 inches and chiseled smooth where required. Beds and vertical joints to be pointed true and full throughout and be laid with $\frac{3}{8}$-inch joints.

The coping is to be doweled as required by the Engineer with round iron,

the dowels to be furnished and placed by the contractor. The drilling for and placing of the dowels will be covered by the price for "Bolt Holes in Masonry." The dowels will be set in Portland cement.

RUBBLE STONE

Quality and Dimensions of.—Rubble stone must be sound, hard, and durable, free from seams, scale, earthy matter, and other defects. Rubble stone shall in general be not less than $\frac{3}{4}$ of a cubic foot in size. It must be in fair shape for laying in the face of the walls without dressing. No spalls will be allowed.

Laying.—The stone must be laid on their natural bed in full beds of hydraulic cement mortar, with all joints and voids well filled with mortar. Leveling up under stones with small chips or spalls will not be allowed.

The stone shall be carefully selected for the outer face so as to have vertical joints and present a good face of broken rough masonry.

CONCRETE

Composition of.—Concrete shall be composed of satisfactory cement and river gravel; the latter, should it be of an approved quality, shall be taken from the various excavations of the lock and its walls. This gravel generally has a sufficient volume of sand to fill all voids; should there be a deficiency of sand in any portion of the gravel the contractor will be required to supply said deficiency by good, sharp, washed, river sand. The quantity of cement to be used will generally be about 20 per cent greater than the volume of voids in sand and gravel.

Mixing and Placing of.—The concrete is to be well and rapidly mixed by machinery, as may be required by the Engineer, unless otherwise specified. It will be deposited in layers not more than 8 inches thick; wherever and whenever required, the layers will be thinner than 8 inches, and all thoroughly rammed by such process as the Engineer may approve.

River Wall.—In the river wall of the lock the concrete shall be laid in courses of a thickness corresponding to the adjoining courses of ashlar masonry. It shall be filled in flush with the top of each course before the next course of ashlar above shall be laid.

Before putting in the concrete of any course the bed and adjoining course of ashlar shall be thoroughly wetted so that no dry surface may come in contact with the fresh concrete, destroying its power of adhesion by absorbing its moisture.

In order that the work once began may progress without delay all cut stone needed for the ashlar facing shall be on the ground when the concrete foundation has been completed.

TIMBER IN PERMANENT CONSTRUCTION

To Consist of all timber used in the timber facing of the lock walls and the guide walls; all timber cribbing in the gate-track and Poirée-dam founda-

tions; the oak sheeting at the head of the guide walls; and such other timber in permanent construction as shall be shown upon the drawings.

General Quality and Dimensions.—All timber must be first class, and any of inferior quality will be rejected. Sap-wood in any stick will cause its rejection. The timber must be free from black or rotten knots, wane edges, wind-shakes, dose, or other imperfections. Firm, sound knots, if not too numerous, will not be considered defects. Timber must be full size, true and out of wind, and when required must be sawed large enough to dress down to required dimensions. The timber will be inspected on arrival at the work, and if found to be defective will be rejected.

Oak.—Oak timber must be taken from the best quality live white oak sawed timber.

White Pine.—Shall consist of the best quality of clear white pine obtainable.

Hemlock.—Shall be the best quality of hemlock obtainable.

Framing, Assembling, and Painting.—All timber must be accurately framed, fitted, and assembled, according to detailed drawings and directions. As the timber is framed it shall be painted about the ends and elsewhere as may be required to prevent checking. The paints for this will be furnished and applied by the contractor, and covered in his price for "Timber in Permanent Construction."

Timber Facing, Uprights, and Sheeting shall be constructed of oak, and shall consist of uprights spaced at intervals of 6 feet, center to center, anchored to the concrete masonry by tee-head screw-bolts as shown on drawings. To the uprights shall be bolted, with wrought-iron screw-bolts, oak sheeting 6 inches thick.

Nosing Timber shall extend along the top of the guide wall, forming a cap to the uprights and securely bolted to them, as shown on the drawings. The top surface of the nosing shall be flush with the top of the concrete masonry wall.

Oak Sheeting.—This refers to the sheeting on the upper faces of the protection crib for the upper guiding wall at the upper end thereof. It shall be spiked on and firmly held in place with iron bands or straps bolted to the framing timbers of the crib, if, in the judgment of the Engineer, this may be deemed necessary.

SUPERVISION AND MEASUREMENT OF WORK

Inspection, Rejected Material, etc.—The works will be conducted under the direction of the local or resident Engineer, who shall have power to prescribe the order and manner of executing the same in all its parts; of inspecting and rejecting materials, work, and workmanship which, in his judgment, do not conform to the drawings that may be furnished from time to time, or to these specifications. And any material, work, or workmanship so rejected by him shall be kept out of or removed from the finished work, and no estimate or payment shall be made until such material, work, or workmanship be so removed.

When so required rejected material shall be piled up in sight near the works and kept there until the Engineer gives permission to have it removed.

The United States will keep inspectors on the work who will receive instructions from the resident Engineer. They will have power to object to any materials, work, or workmanship. Any material, work, or workmanship objected

to by the inspectors shall be kept out of or removed from the finished work, unless in each particular case the objections of the inspector shall be overruled by the local or resident Engineer; and, unless the objection be so overruled, no estimate or payment shall be made until such material, work, or workmanship be so removed.

The local or resident Engineer shall have power to overrule or rescind any or all objections or decisions of the inspector.

The decision of the United States Engineer Officer in charge of the works shall be final and conclusive upon all matters relating to the work and upon all questions arising out of these specifications, and from his decision there shall be no appeal.

Failure to Prosecute or Protect Works.—If at any time the contractor shall refuse or fail to prosecute the work or provide for carrying on the same as directed by the Engineer, or fail to properly protect any part of the work, permanent or temporary, the Engineer shall have power to employ men, to purchase or otherwise provide materials, tools, machinery, etc., and put the work in proper advancement or condition, and the entire cost of so doing shall be deducted from payments to be made under this contract.

Complete Work Required.—The contractor is not to take advantage of any omissions of details in drawings or specifications, or errors in either, but he will be required to do everything which may be necessary to carry out the contract in good faith, which contemplates everything complete, in good working order, of good material, with accurate workmanship, skillfully fitted and properly connected and put together. Any point not clearly understood is to be referred to the Engineer for decision.

Changes.—Should any changes in the details of the shape, arrangement, or fitting of the parts be deemed necessary or advisable in the progress of the work, they must be made by the contractor, and a fair allowance will be paid for any changes which, in the judgment of the Engineer in charge, materially increase the cost of the work.

Measurement.—Measurement of all work and material shall be made in place, unless otherwise specified.

Coffer-dam.—The price per lineal foot of coffer-dam shall include all material, lumber, iron, and gravel entering into its construction. A profile of the location will be furnished, showing a section of the river-bed over which the coffer-dam is located, so that the contractor may estimate the amount of each kind of material required.

Piling.—Temporary piling shall be measured in lineal feet, and measurement shall be allowed for total length of piling used.

Sheeting.—This will include all lumber used for temporary purposes, in shoring of excavations, or for forms necessary to sustain any concrete masonry until it has become sufficiently hardened. Sheeting required by the Engineer to remain in the finished structure shall be paid for at the contractor's price per thousand feet B.M. All temporary sheeting not remaining in the finished structure shall be included in the contractor's unit price for material in place, and no estimate will be made thereof by the Engineer. Coffer-dam sheeting will be included in the contractor's price per lineal foot of coffer-dam.

Filling.—Gravel filling will be measured in the fill, and will not include any filling placed in the coffer-dam as coffer-dam filling.

Stone filling shall include all riprap work, either temporary or permanent.

EXCAVATION.—Excavation will be measured in excavation by cross-sections.

MASONRY.—All masonry, ashlar, rubble, brick, concrete, etc., will be measured by the cubic yard in place. Prices for masonry will include all required pointing. No payment will be allowed for voids or openings.

BOLT HOLES.—All holes drilled in rock or concrete or other masonry will be measured by the running foot as drilled.

TIMBER IN PERMANENT CONSTRUCTION.—Timber in permanent construction will include all timber used in any part of the permanent construction; unless otherwise particularly specified, it will be classed under the following heads:

Oak in Permanent Construction.
Pine in Permanent Construction.
Hemlock in Permanent Construction.

APPENDIX II

EXTRACTS FROM TOPEKA (KANSAS) MELAN ARCH BRIDGE SPECIFICATIONS

By permission of EDWIN THACHER, M. Am. Soc. C. E.

PILING IN PERMANENT WORK

Piling and lumber for coffer-dams to be sound white oak, yellow pine, or other woods equally good for the purpose, the quality to be acceptable to the superintendent. The piles shall be straight-grained, trimmed close, and have all bark taken off, and shall be at least 10 inches in diameter at the small end and 14 inches in diameter at the butt when sawed off. The heads shall be cut off squarely at right angles to the axis of the pile, and all piles shall be fitted to and driven with a cast-iron head. The piles shall be driven with a hammer weighing not less than two thousand two hundred and fifty (2250) pounds, and until they do not move more than three-eighths ($\frac{3}{8}$) of an inch under a blow of the hammer falling twenty-five (25) feet. No pile shall be driven less than twenty-six (26) feet below low water, and if necessary to attain this minimum depth jets shall be used in addition to hammer. The number and arrangement of the piles for each foundation are shown on the plans, and must be carefully carried out by the contractor. The piles shall be cut off at an elevation of about six (6) inches below low water. A slight variation will be allowed, but no piles must be cut off at a higher elevation. Inspection of piling and lumber, except at bridge site, shall be at contractor's expense.

COFFER-DAMS

After the bearing piles have been driven, a permanent coffer-dam, of the dimensions marked on the plans, of Wakefield (or other equally satisfactory) sheet-piling, shall be used around each foundation. The earth inside thereof shall be excavated to the depth shown on plans and replaced with concrete as hereinafter specified. During the placing of the concrete the water shall be kept out of the coffer-dams, unless the bottom is so porous that it is impracticable in the opinion of the superintendent to do so—in which case some of the concrete may be placed in position by means of chutes, under the direction of the superintendent, until the bottom is well calked, after which the water shall be pumped out and the remaining concrete placed in position. The contractor will be required to make the sides and ends of the coffer-dams watertight, and no leak through them will be considered sufficient cause to require any concrete to be placed by means of chutes.

CENTERING

The contractor shall build an unyielding falsework, or centering, of the form and dimensions shown on the plans; particular care must be taken to drive the piles supporting it to a solid bearing. The estimated load upon each of these piles is twenty (20) tons. The contactor must, however, satisfy himself as to the load each pile will have to bear, and as to its supporting power. In case of any settlement the contractor shall take down and rebuild the centering and arch. The lagging shall be dressed on both edges to a uniform size, so that when laid it will present a smooth surface, and this surface shall be built at the proper elevation to allow for settlement of arch, so that when the centering is struck the arch-ring will come to the elevations shown on plans.

The top surface of the lagging shall be covered with W. Field's Building Paper of medium weight, known as Double Saturated Water-proof Oiled Sheathing Paper (or other equally good) to prevent the concrete from adhering thereto. No center shall be struck until at least twenty-eight (28) days after the completion of the arch. Great care shall be used in lowering the centers so as not to throw undue strains upon the arches, nor shall any center be struck before the adjoining arch has been completed for a sufficiently long time, in the opinion of the superintendent, to be uninjured thereby.

NOTE.—For the above reasons it is probable that the five centers will be in use at the same time.

PORTLAND CEMENT

The Portland cement shall be a true Portland cement, made by calcining a proper mixture of calcareous and clayey earths; and the contractor shall furnish one or more certified statements of the chemical composition of the cement and of the raw materials from which it is manufactured. Only one brand of Portland cement shall be used on the work, except with permission of the superintendent, and it shall in no case contain more than two (2) per cent of magnesia in any form.

The fineness of the cement shall be such that at least 98 per cent shall pass through a standard brass cloth sieve of 74 meshes per linear inch, and at least 95 per cent shall pass through a sieve of 100 meshes per linear inch.

Samples for testing may be taken from each and every barrel delivered, as superintendent may direct. Tensile tests will be made on specimens prepared and maintained, until tested, at a temperature of not less than 60° Fahrenheit. Each specimen shall have an area of one square inch at the breaking section, and after being allowed to harden in moist air for twenty-four hours shall be immersed and retained under water until tested.

The sand used in preparing the test specimens shall be clean, sharp, crushed quartz, retained on a sieve of 30 meshes per linear inch and passed through a seive of 20 meshes per linear inch, and shall be furnished by contractor.

No more than 23 to 27 per cent of water by weight shall be used in preparing the test specimens of neat cement, and in making the test specimens one of cement to three of sand, no more than 11 or 12 per cent of water by weight shall be used.

Specimens prepared from neat cement shall after seven days develop a tensile strength not less than 400 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand (parts by weight) shall after seven days develop a tensile strength of not less than 140 pounds per square inch, and after twenty-eight days not less than 200 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand (parts by weight) and immersed, after twenty-four hours, in water to be maintained at 176° Fahrenheit, shall not swell nor crack, and shall after seven days develop a tensile strength of not less than 140 pounds per square inch.

Cement mixed neat with about 27 per cent. of water, to form a stiff paste, shall, after 30 minutes, be appreciably indented by the end of a wire one-twelfth inch in diameter, loaded to weigh one-quarter pound.

Cement made into thin cakes on glass plates shall not crack, scale, or warp under the following treatment. Three pats shall be made and allowed to harden in moist air at from 60° to 70° Fahrenheit; one of these shall be subjected to water-vapor at 176° Fahrenheit for three hours, after which it shall be immersed in hot water for forty-eight hours; another shall be placed in water at from 60° to 70° Fahrenheit, and the third shall be left in moist air.

Samples of one-to-two mortar and of concrete shall be made and tested from time to time as directed by the superintendent. All cement shall be housed and kept dry till wanted in the work.

Storage rooms and rooms and apparatus for the tests shall be furnished by the contractor, and all tests shall be made entirely at his expense, and under the direction and to the satisfaction of the superintendent.

PORTLAND CEMENT CONCRETE

The concrete shall be composed of clean, hard, broken limestone (or gravel with irregular surfaces) and cement mortar in volumes as hereinafter described. The sand shall be clean, sharp, Kansas River sand, washed *entirely* free from earth and loam. If obtainable, a mixture of coarse and fine sand shall be used. Approved mixing machines shall be used. These machines must be kept clean and no accumulations of old mortar shall be allowed to form in them. The ingredients shall be placed in the machine in a dry state and in the volumes specified and be throughly mixed, after which clean water shall be added and the mixing continued until the wet mixture is thorough and the mass uniform. No more water shall be used than the concrete will bear without quaking in ramming. The mixing must be done as rapidly as possible, and the batch deposited in the work without delay, and before the cement begins to set. Stone must be entirely free from earth and earthy surfaces. Thin splints or leaves of stone, easily broken with fingers, will not be allowed to go into the work. The quality of stone and the crushing must be acceptable to the superintendent.

The grades of concrete to be used are as follows (parts by volume):

For the arches: One part Portland cement, two parts sand, and four parts broken stone (hazelnut size, from one-half inch to one inch), except for the exposed faces and soffits of the arches, which shall have at least one inch in thickness of mortar composed of one part Portland cement and two parts sand.

For the piers, abutments, spandrel and wing-walls: On the exposed surfacer for at least one inch thick, one part Portland cement and two parts sand; for

the next seven (7) inches, one part Portland cement, two parts sand, and four parts broken stone of hazelnut size. For the remaining portions: One part Portland cement, four parts sand, and eight parts broken stone of size to pass through a 3-inch ring, except such portions of the interior of the piers and abutments as are above the top of the cornice, or elevation 15.75 feet above low water, which shall be composed of one part Portland cement, three parts sand, and six parts broken stone which will pass through a 2½-inch ring.

No plastering of surfaces will be allowed nor any practice that will develop planes or surfaces of demarcation other than those hereinafter described. Immediately after the removal of any forms or centers, sand and cement shall be sifted on the surfaces and the surfaces rubbed hard with a float as may be directed by the superintendent.

During warm and dry weather and whenever the superintendent shall direct, all newly built concrete shall be kept well shaded from the sun and well sprinkled with water at the surface for several days or until well set.

There must be no definite plane or surface of demarcation between the facing and the concrete backing. The facing and the backing must be deposited in the same layer and well rammed in place at the same time.

In connecting old concrete with new, in the planes hereafter described, the old concrete shall be cleaned and roughened and soaked with water, and at the points of contact a mortar composed of one part cement and two parts sand shall be used and shall be laid in the same manner as specified for laying the facing.

NATURAL CEMENT CONCRETE

The concrete around piles, to take the place of the earth excavated from the coffer-dams, shall be composed of one part natural cement, equal to the best Fort Scott, Kas., cement, three parts sand, and six parts of broken stone of the size to pass through a 3-inch ring. This concrete may be mixed by hand on platforms adjoining the foundations and shoveled directly into the coffer-dams, care being taken to deposit it in uniform layers of about 6 inches each and to carefully ram each layer.

PIERS, ABUTMENTS, AND SPANDRELS

All piers, abutments, spandrels, and wing-walls shall be built in timber forms. These forms shall be substantial and unyielding, of proper dimensions for the work intended and closely jointed, and all surfaces that come in contact with the concrete shall be smoothly dressed and well oiled with linseed oil to prevent the concrete from adhering to them. That portion next to the exposed faces of the work need not be oiled, but shall be covered with oiled paper, the same as that specified for the centers.

Molds, to form molding and panels, smoothly finished and well oiled with linseed oil, shall be properly placed in the forms so that the finished work will appear as shown on the plans. Extreme care must be used to place them in proper position before placing any concrete or mortar in them.

CONTINUOUS WORK

The following divisions shall constitute sections for continuous work, viz.: Each footing course of piers or abutments; each pier or abutment from footing course to cornice; each pier or abutment from cornice to springing line of arch; each spandrel wall from keystone to pier or abutment; each pier or abutment spandrel wall; that portion of the piers or abutments above springing line of arch shall be considered part of the longitudinal sections of the arch previously described.

Each of the above sections shall be carried on continuously night and day if necessary; that is, each layer shall be well rammed in place before the previously deposited layer shall have time to partially set.

Care shall be taken to make the joints (for expansion) in each spandrel wall over piers as indicated on the plans.

CONCRETE IN COFFER-DAMS

The natural cement concrete in the coffer-dam shall extend from depths marked on plans to 1 foot below low water. Upon this concrete the footing courses of piers and abutments shall be founded.

The sheet-piling of coffer-dams shall be cut off at least down to low-water mark, neatly and evenly, by the contractor before the completion of the work.

APPENDIX III

EXTRACTS FROM KATTE'S MASONRY SPECIFICATIONS

By permission of WALTER KATTE, M. Am. Soc. C.E.

EXCAVATIONS will be classified under the following heads, viz.: Earth, hardpan, loose rock, solid rock, and excavation in water.

EARTH will include clay, sand, gravel, loam, decomposed rock and slate, stones and boulders containing less than one cubic foot, and all other matters of an earthy nature, however compact, excepting only " hardpan," as described below.

HARDPAN will consist of tough, indurated clay or cemented gravel which, in the opinion of the Engineer, requires blasting for its removal.

LOOSE ROCK.—All boulders and detached masses of rock measuring over one (1′) cubic foot in bulk, and less than one (1) cubic yard; also all slate, shale, soft friable sandstone and soapstone, and all other materials excepting rock, solid ledge, and those described above; also stratified rock in layers of not exceeding eight (8″) inches in thickness, when separated by strata of clay, and which, in the judgment of the Engineer, may be removed without blasting, although blasting may occasionally be resorted to.

SOLID ROCK will include all rock found in ledges, or masses of more than one (1) cubic yard, which, in the judgment of the Engineer, may be best removed by blasting, with the exception of stratified rocks described under the head of Loose Rock. In rock excavations the " bottom " must in all cases be taken down truly to sub-grade; and when so ordered by the Engineer ditches must be formed at the foot of the slope.

The contract price for excavations will apply to pits required for foundations of masonry when water is not encountered, and the price for

EXCAVATION IN WATER will only apply to foundation pits under water and deepening of channels in running water; it must cover all classes of material, and include drainage, bailing, pumping, and all materials and labor connected with such excavations, also the necessary dressing of the rock.

CEMENT must be of the best quality of freshly burned and ground hydraulic cement, and be equal in quality to the best brands of
It will be subject to test made by the Engineer or his appointed inspector, and must stand a proof tensile test of fifty (50) pounds per square inch of sectional area on specimens allowed a set of thirty (30) minutes in air and twenty-four (24) hours under water.

MORTAR will in all cases be made of one part in bulk of the best hydraulic cement to two parts in bulk of clean, sharp sand, well and thoroughly mixed together in a clean box of boards, before the addition of the water, and must

be used immediately after being mixed. No mortar left overnight will, under any pretext, be allowed to be used. The sand and cement used will at all times be subject to inspection, test, acceptance, or rejection by the Engineer.

CONCRETE.—Concrete shall be composed of fragments of hard, sound and acceptable stone, broken to a size that will pass through a two (2″) inch ring in any direction, thoroughly clean and free from mud, dust, dirt, or any earthy admixture whatever; mixed in the proportion of two (2) parts in bulk of the broken stone to one (1) part of fresh-made cement mortar of the quality herein described; and is to be quickly laid in sections and in layers not exceeding nine (9) inches in thickness, and to be thoroughly rammed until the mortar flushes to the surface; it shall be allowed at least twelve (12) hours to "set" before any work is laid on it.

FOUNDATIONS

GENERAL DESCRIPTION.—Foundations for masonry shall be excavated to such depths as may be necessary to secure a solid bearing for the masonry, of which the Engineer shall be the judge. The materials excavated will be classified and paid for, as provided for in these specifications, under the general head of Excavations; and in case of foundations in rock, the rock must be excavated to such depth and in such form as may be required by the Engineer, and must be dressed level to receive the foundation course.

When a safe and solid foundation for masonry cannot be found at a reasonable depth (to be judged of by the Engineer), there will be prepared by the contractor such artificial foundations as the Engineer may direct. All materials taken from the excavations for foundations, if of proper quality, shall be deposited in the contiguous embankment; but any material unfit for such purpose shall be deposited outside the roadway, or in such place as the Engineer shall direct, and so that it shall not interfere with any drain or water-course.

TIMBER.—Timber foundations when required shall be such as the Engineer may by drawings or otherwise prescribe, and will be paid for by the one thousand feet, board measure. The price, covering cost of material, framing, and putting in place, and all wrought- and cast-iron work ordered by the Engineer, will be paid for per pound, the price including cost of material, manufacture, and placing in the work.

PILING.—All timber used in foundations or foundation piling shall be of young, sound, and thrifty white oak, yellow pine, or other timber equally good for the purpose, acceptable to the Engineer. Piles must be at least eight (8″) inches in diameter at the small end and twelve (12″) inches in diameter at the butt when sawn off; they must be perfectly straight and be trimmed close, and have the bark stripped off before they are driven. They must be driven into hard bottom until they do not move more than one-half inch under the blow of a hammer weighing two thousand (2000) pounds, falling twenty-five (25′) feet at the last blow. They must be driven vertically and at the regular distances apart from centers, transversely and longitudinally as required by the plans or directions of the Engineer; they must be cut off squarely at the butt and be well sharpened to a point, and when necessary, in the opinion of the Engineer, shall be shod with iron and the heads bound with iron hoops, of such dimensions as he may direct, which will be paid for the same as other iron work used in foundations.

The necessary length of piles shall be ascertained by driving test piles in different parts of the localities in which they are to be used; and in case a pile shall not prove long enough to reach " hard bottom " it shall be sawed off square, and a hole two (2″) inches in diameter be bored into its head twelve (12″) inches deep; into this hole a circular white-oak trenail twenty-three (23″) inches in length shall be well driven, and another pile similarly squared and bored, and of as large a diameter at the small end as can be procured, shall be placed upon the lower pile, brought to its proper position, and driven as before directed. All piles, when thus driven to the required depth, are to be cut off truly square and horizontal at the proper height given by the Engineer, and only the actual number of lineal feet of the piles left for use in the foundations after being sawn off will be paid for.

Coffer-dams.—Where coffer-dams are, in the opinion of the Engineer, required for foundations the prices provided in the contract for timber, piles, and iron in foundations will be allowed for the material and work on same, which is understood as covering all risks from high water or otherwise, draining, bailing, pumping, and all materials connected with the coffer-dams. Sheet-piling will be classed as plank in foundations, and will be paid for per one thousand (1000′) feet, board measure, if left in the ground.

TIMBER

All timber must be sound, straight-grained, and free from sap, loose or rotten knots, wind-shakes, or any other defect that would impair its strength or durability; it must be sawed (or hewed) perfectly straight and to exact dimensions, with full corners and square edges; all framing must be done in a thoroughly workmanlike manner, and both material and workmanskip will be subject to the inspection and acceptance of the Engineer.

APPENDIX IV

SPECIFICATIONS FOR STEEL COFFER-DAM

DESIGN.—The shell shall be made of elliptical shape for ordinary piers and circular for pivot piers. It shall be made not less than four feet larger than footing of pier in plan, to allow for variation in position during sinking.

The plates used shall be as large as can be handled with ease in the shop, during shipment, and during erection.

The splices may be either lap or butt joints, provided a good tight job will result, and the rivets must be spaced according to boiler-maker's rules.

The joint may be made tight by calking or by the use of a calking strip, but in either event the result must be guaranteed.

The shell must be stiffened by horizontal stiffening angles, girders, or trussing, to resist deformation during the placing and to resist both the quiescent and a maximum unbalanced earth or water pressure, or a wind pressure.

The bottom plates shall be re-enforced with narrow plates inside and outside, to form a wedge-shaped cutting edge; and when there is rock or hard bottom, the plates shall be cut to conform to its contour as nearly as possible.

The top shall be properly stiffened, and if necessary provided with connection holes for additional sections.

The factor for safety shall in no case be less than four, and in case the shell will be subject to shock, not less than six.

No metal of a less thickness than $\frac{1}{4}$ inch shall be used for temporary work, nor less than $\frac{3}{8}$ inch for permanent work in fresh water or $\frac{1}{2}$ inch in salt water.

MATERIAL.—The entire shell shall be constructed of the grade of steel known as soft medium, except rivets, which shall be of bridge quality of iron.

The steel may be made either by the Bessemer or open-hearth process, and the phosphorus shall never exceed 0.08 per cent.

Soft medium steel shall have an ultimate strength of from 55,000 to 65,000 pounds per square inch, as determined from standard test pieces; an elastic limit of not less than one-half the ultimate strength; an elongation of not less than 25 per cent in 8 inches; and a reduction of area at fracture of not less than 50 per cent.

Samples to bend cold 180° to a diameter equal to the thickness of the sample, without crack or flaw on the outside of the bent portion.

ERECTION.—The erection must be done in a first-class manner, and all rivets must have full heads. The shell shall be placed in position within one-half the distance allowed for error in the design of the coffer-dam. Only a reasonable variation will be allowed for difference in level.

Painting.—All the metal work shall be thoroughly cleaned of rust or scale at the shops and coated thoroughly with hot asphaltum.

Before erection, in the field, it shall be given a second coating of hot asphaltum.

Sealing.—When in position on the bottom, if the coffer-dam has not been sunk through impervious strata, it shall be sealed by concreting around the circumference inside with concrete passed through a tube.

Removal.—Should the coffer-dam not form a part of the permanent foundation it shall be taken apart, at the joints designed for the purpose, and carefully removed in such a manner as not to injure the foundation, and so as to be used again if required.

APPENDIX V

STANDARD SPECIFICATIONS AND TESTS FOR PORTLAND CEMENT

American Society for Testing Materials

SERIAL DESIGNATION: C 9–17

These specifications and tests are issued under the fixed designation C 9; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision.

Adopted, 1904; Revised, 1908, 1909, 1916 (Effective Jan. 1, 1917)

SPECIFICATIONS

1. DEFINITION. Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportional mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

I. CHEMICAL PROPERTIES

2. CHEMICAL LIMITS. The following limits shall not be exceeded:

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulphuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

II. PHYSICAL PROPERTIES

3. SPECIFIC GRAVITY. The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered.

4. FINENESS. The residue on a standard No. 200 sieve shall not exceed 22 per cent by weight.

5. SOUNDNESS. A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness.

6. TIME OF SETTING. The cement shall not develop initial set in less than forty-five minutes when the Vicat needle is used or sixty minutes when the Gillmore needle is used. Final set shall be attained within ten hours.

7. TENSILE STRENGTH. The average tensile strength in pounds per square inch of not less than three standard mortar briquettes (see Section 51) composed

of one part cement and three parts standard sand, by weight, shall be equal to or higher than the following:

Age at Test, Days.	Storage of Briquettes.	Tensile Strength, Lb. per Sq. In.
7	1 day in moist air, 6 days in water	200
28	1 day in moist air, 27 days in water	300

8. The average tensile strength of standard mortar at twenty-eight days shall be higher than the strength at seven days.

III. Packages, Marking and Storage

9. Packages and Marking. The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacture plainly marked thereon, unless shipped in bulk. A bag shall contain 94 pounds net. A barrel shall contain 376 pounds net.

10. Storage. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness.

IV. Inspection

11. Inspection. Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least ten days from the time of sampling shall be allowed for the completion of the seven-day test, and at least thirty-one days shall be allowed for the completion of the twenty-eight day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The twenty-eight day test shall be waived only when specifically so ordered.

V. Rejection

12. Rejection. The cement may be rejected if it fails to meet any of the requirements of these specifications.

13. Cement shall not be rejected on account of failure to meet the fineness requirement if upon retest after drying at 100° C. for one hour it meets this requirement.

14. Cement failing to meet the test for soundness in steam may be accepted if it passes a retest using a new sample at any time within twenty-eight days thereafter.

15. Packages varying more than 5 per cent from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

TESTS

VI. Sampling

16. Number of Samples. Tests may be made on individual or composite samples as may be ordered. Each test sample should weigh at least 8 pounds.

17. (*a*) *Individual Sample.* If sampled in cars one test sample shall be taken from each 50 barrels or fraction thereof. If sampled in bins one sample shall be taken from each 100 barrels.

(*b*) *Composite Sample.* If sampled in cars one sample shall be taken from one sack in each 40 sacks (or 1 barrel in each 10 barrels) and combined to form one test sample. If sampled in bins or warehouses one test sample shall represent not more than 200 barrels.

18. Method of Sampling. Cement may be sampled at the mill by any of the following methods that may be practicable, as ordered:

(*a*) *From the Conveyer Delivering to the Bin.* At least 8 pounds of cement shall be taken from approximately each 100 barrels passing over the conveyor.

(*b*) *From Filled Bins by Means of Proper Sampling Tubes.* Tubes inserted vertically may be used for sampling cement to a maximum depth of 10 feet. Tubes inserted horizontally may be used where the construction of the bin permits. Samples shall be taken from points well distributed over the face of the bin.

(*c*) *From Filled Bins at Points of Discharge.* Sufficient cement shall be drawn from the discharge openings to obtain samples representative of the cement contained in the bin, as determined by the appearance at the discharge openings of indicators placed on the surface of the cement directly above these openings before drawing of the cement is started.

19. Treatment of Sample. Samples preferably shall be shipped and stored in air-tight containers. Samples shall be passed through a sieve having 20 meshes per linear inch in order to thoroughly mix the sample, break up lumps and remove foreign materials.

VII. Chemical Analysis

Loss on Ignition

20. Method. One gram of cement shall be heated in a weighed covered platinum crucible, of 20 to 25 cc. capacity, as follows, using either method (*a*) or (*b*) as ordered.

(*a*) The crucible shall be placed in a hole in an asbestos board, clamped horizontally so that about three-fifths of the crucible projects below, and blasted at a full red heat for fifteen minutes with an inclined flame; the loss in weight shall be checked by a second blasting for five minutes. Care shall be taken to wipe off particles of asbestos that may adhere to the crucible when withdrawn from the hole in the board. Greater neatness and shortening of the time of heating are secured by making a hole to fit the crucible in a circular disk of sheet platinum and placing this disk over a somewhat larger hole in an asbestos board.

(*b*) The crucible shall be placed in a muffle at any temperature between 900 and 1000° C. for fifteen minutes and the loss in weight shall be checked by a second heating for five minutes.

21. Permissible Variation. A permissible variation of 0.25 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 4 per cent.

Insoluble Residue

22. Method. To a 1-gram sample of cement shall be added 10 cc. of water and 5 cc. concentrated hydrochloric acid; the liquid shall be warmed until effervescence ceases. The solution shall be diluted to 50 cc. and digested on a steam

bath or hot plate until it is evident that decomposition of the cement is complete. The residue shall be filtered, washed with cold water, and the filter-paper and contents digested in about 30 cc. of a 5 per cent solution of sodium carbonate, the liquid being held at a temperature just short of boiling for fifteen minutes. The remaining residue shall be filtered, washed with cold water, then with a few drops of hot hydrochloric acid, 1:9, and finally with hot water, and then ignited at a red heat and weighed as the insoluble residue.

23. PERMISSIBLE VARIATION. A permissible variation of 0.15 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 0.85 per cent.

SULPHURIC ANHYDRIDE

24. METHOD. One gram of the cement shall be dissolved in 5 cc. of concentrated hydrochloric acid diluted with 5 cc. of water, with gentle warming; when solution is complete 40 cc. of water shall be added, the solution filtered, and the residue washed thoroughly with water. The solution shall be diluted to 250 cc., heated to boiling and 10 cc. of a hot 10 per cent solution of barium chloride shall be added slowly, drop by drop, from a pipette and the boiling continued until the precipitate is well formed. The solution shall be digested on the steam bath until the precipitate has settled. The precipitate shall be filtered, washed, and the paper and contents placed in a weighed platinum crucible and the paper slowly charred and consumed without flaming. The barium sulphate shall then be ignited and weighed. The weight obtained multiplied by 34.3 gives the percentage of sulphuric anhydride. The acid filtrate obtained in the determination of the insoluble residue may be used for the estimation of sulphuric anhydride instead of using a separate sample.

25. PERMISSIBLE VARIATION. A permissible variation of 0.10 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 2.00 per cent.

MAGNESIA

26. METHOD. To 0.5 gram of the cement in an evaporating dish shall be added 10 cc. of water to prevent lumping and then 10 cc. of concentrated hydrochloric acid. The liquid shall be gently heated and agitated until attack is complete. The solution shall then be evaporated to complete dryness on a steam or water bath. To hasten dehydration the residue may be heated to 150 or even 200° C. for one-half to one hour. The residue shall be treated with 10 cc. of concentrated hydrochloric acid diluted with an equal amount of water. The dish shall be covered and the solution digested for ten minutes on a steam bath or water bath. The diluted solution shall be filtered and the separated silica washed thoroughly with water.* Five cubic centimeters of concentrated hydrochloric acid and sufficient bromine water to precipitate any manganese which may be present shall be added to the filtrate (about 250 cc.). This shall be made alkaline with ammonium hydroxide, boiled until there is but a faint odor of ammonia, and the precipitated iron and aluminum hydroxides, after settling, shall be washed with hot water, once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate shall be transferred by a jet of hot water to the precipitating vessel

* Since this procedure does not involve the determination of silica, a second evaporation is unnecessary.

and dissolved in 10 cc. of hot hydrochloric acid. The paper shall be extracted with acid, the solution and washings being added to the main solution. The aluminum and iron shall then be reprecipitated at boiling heat by ammonium hydroxide and bromine water in a volume of about 100 cc., and the second precipitate shall be collected and washed on the filter used in the first instance if this is still intact. To the combined filtrates from the hydroxides of iron and aluminum, reduced in volume if need be, 1 cc. of ammonium hydroxide shall be added, the solution brought to boiling, 25 cc. of a saturated solution of boiling ammonium oxalate added, and the boiling continued until the precipitated calcium oxalate has assumed a well-defined granular form. The precipitate after one hour shall be filtered and washed, then with the filter shall be placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner; after ignition it shall be redissolved in hydrochloric acid and the solution diluted to 100 cc. Ammonia shall be added in slight excess, and the liquid boiled. The lime shall then be reprecipitated by ammonium oxalate, allowed to stand until settled, filtered and washed. The combined filtrates from the calcium precipitates shall be acidified with hydrochloric acid, concentrated on the steam bath to about 150 cc. and made slightly alkaline with ammonium hydroxide, boiled and filtered (to remove a little aluminum and iron and perhaps calcium). When cool, 10 cc. of saturated solution of sodium-ammonium-hydrogen phosphate shall be added with constant stirring. When the crystalline ammonium-magnesium orthophosphate has formed, ammonia shall be added in moderate excess. The solution shall be set aside for several hours in a cool place, filtered and washed with water containing 2.5 per cent of NH_3. The precipitate shall be dissolved in a small quantity of hot hydrochloric acid, the solution diluted to about 100 cc., 1 cc. of a saturated solution of sodium-ammonium-hydrogen phosphate added, and ammonia drop by drop, with constant stirring, until the precipitate is again formed as described and the ammonia is in moderate excess. The precipitate shall then be allowed to stand about two hours, filtered and washed as before. The paper and contents shall be placed in a weighed platinum crucible, the paper slowly charred, and the resulting carbon carefully burned off. The precipitate shall then be ignited to constant weight over a Meker burner, or a blast not strong enough to soften or melt the pyrophosphate. The weight of magnesium pyrophosphate obtained multiplied by 72.5 gives the percentage of magnesia. The precipitate so obtained always contains some calcium and usually small quantities of iron, aluminum, and manganese as phosphates.

27. Permissible Variation. A permissible variation of 0.4 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 5.00 per cent.

VIII. Determination of Specific Gravity

28. Apparatus. The determination of specific gravity shall be made with a standardized Le Chatelier apparatus which conforms to the requirements illustrated in Fig. 273. This apparatus is standardized by the United States Bureau of Standards. Kerosene free from water, or benzine not lighter than 62° Baumé, shall be used in making this determination.

29. Method. The flask shall be filled with either of these liquids to a point on the stem between zero and 1 cc., and 64 grams of cement, of the same temperature as the liquid, shall be slowly introduced, taking care that the cement

does not adhere to the inside of the flask above the liquid and to free the cement from air by rolling the flask in an inclined position. After all the cement is

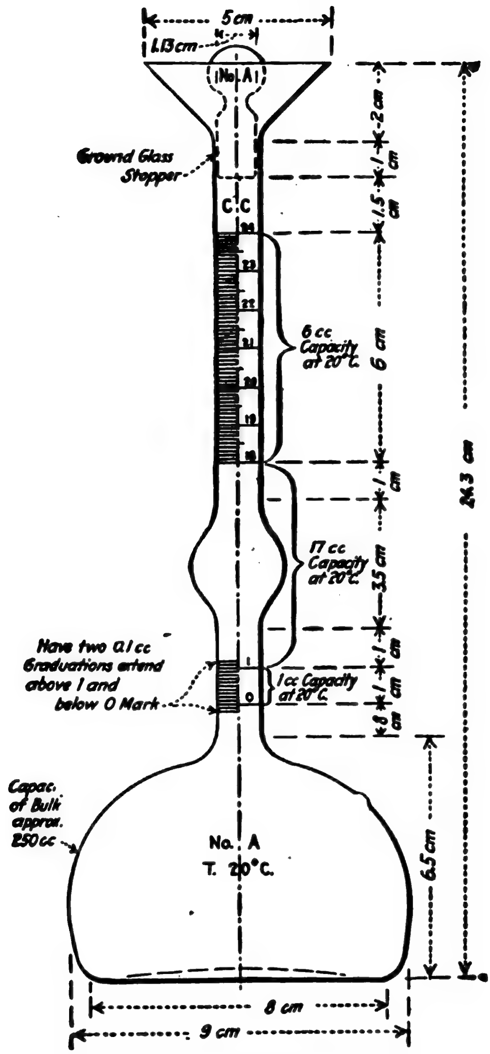


FIG. 273.—LE CHATELIER APPARATUS

introduced, the level of the liquid will rise to some division of the graduated neck; the difference between readings is the volume displaced by 64 grams of the cement.

The specific gravity shall then be obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement (g.)}}{\text{Displaced volume (cc.)}}.$$

30. The flask, during the operation, shall be kept immersed in water, in order to avoid variations in the temperature of the liquid in the flask, which shall not exceed 0.5° C. The results of repeated tests should agree within 0.01.

31. The determination of specific gravity shall be made on the cement as received; if it falls below 3.10, a second determination shall be made after igniting the sample as described in Section 20.

IX. Determination of Fineness

32. Apparatus. Wire cloth for standard sieves for cement shall be woven (not twilled) from brass, bronze, or other suitable wire, and mounted without distortion on frames not less than 1½ inches below the top of the frame. The sieve frames shall be circular, approximately 8 inches in diameter, and may be provided with a pan and cover.

33. A standard No. 200 sieve is one having nominally an 0.0029-inch opening and 200 wires per inch standardized by the U. S. Bureau of Standards, and conforming to the following requirements:

The No. 200 sieve should have 200 wires per inch, and the number of wires in any whole inch shall not be outside the limits of 192 to 208. No opening between adjacent parallel wires shall be more than 0.0050 inch in width. The diameter of the wire should be 0.0021 inch and the average diameter shall not be outside the limits 0.0019 to 0.0023 inch. The value of the sieve as determined by sieving tests made in conformity with the standard specification for these tests on a standardized cement which gives a residue of 25 to 20 per cent on the No. 200 sieve, or on other similarly graded material, shall not show a variation of more than 1.5 per cent above or below the standards maintained at the Bureau of Standards.

34. Method. The test shall be made with 50 gram of cement. The sieves shall be thoroughly clean and dry. The cement shall be placed on the No. 200 sieve, with pan and cover attached, if desired, and shall be held in one hand in a slightly inclined position so that the sample will be well distributed over the sieve, at the same time gently striking the side about 150 times per minute against the palm of the other hand on the upstroke. The sieve shall be turned every 25 strokes about one-sixth of a revolution in the same direction. The operation shall continue until not more than 0.05 gram passes through in one minute of continuous sieving. The fineness shall be determined from the weight of the residue on the sieve expressed as a percentage of the weight of the original sample.

35. Mechanical sieving devices may be used, but the cement shall not be rejected if it meets the fineness requirement when tested by the hand method described in Section 34.

36. Permissible Variation. A permissible variation of 1 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 22 per cent.

X. Mixing Cement Pastes and Mortars

37. Method. The quantity of dry material to be mixed at one time shall not exceed 1000 grams nor be less than 500 grams. The proportions of cement or cement and sand shall be stated by weight in grams of the dry materials; the quantity of water shall be expressed in cubic centimeters (1 cc. of water = 1 gram). The dry materials shall be weighed, placed upon a non-absorbent surface, thoroughly

mixed dry if sand is used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the crater by the aid of a trowel. After an interval of one-half minute for the absorption of the water the operation shall be completed by continuous, vigorous mixing, squeezing and kneading with the hands for at least one minute.* During the operation of mixing, the hands should be protected by rubber gloves.

38. The temperature of the room and the mixing water shall be maintained as nearly as practicable at 21° C. (70° F.).

XI. Normal Consistency

39. Apparatus. The Vicat apparatus consists of a frame *A* (Fig. 274) bearing a movable rod *B*, weighing 300 grams, one end *C* being 1 cm. in diameter

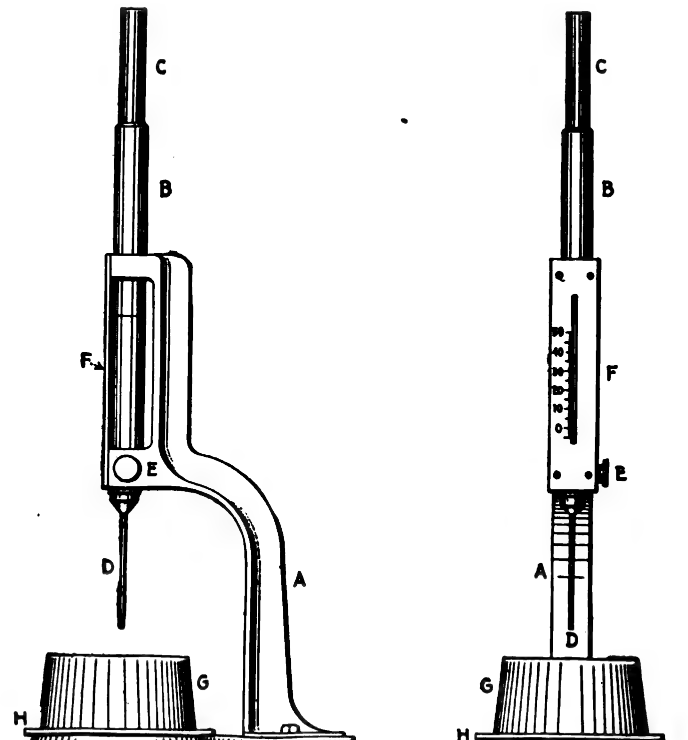


Fig. 274.—Vicat Apparatus

for a distance of 6 cm., the other having a removable needle *D*, 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in any desired position by a

* In order to secure uniformity in the results of tests for the time of setting and tensile strength the manner of mixing above described should be carefully followed. At least one minute is necessary to obtain the desired plasticity which is not appreciably affected by continuing the mixing for several minutes. The exact time necessary is dependent upon the personal equation of the operator. The error in mixing should be on the side of over mixing.

screw E, and has midway between the ends a mark F which moves under a scale (graduated to millimeters) attached to the frame A. The paste is held in a conical, hard-rubber ring G, 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate H about 10 cm. square.

40. METHOD. In making the determination, 500 grams of cement, with a measured quantity of water, shall be kneaded into a paste, as described in Section 37, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 inches apart; the ball resting in the palm of one hand shall be pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end shall then be removed by a single movement of the palm of the hand; the ring shall then be placed on its larger end on a glass plate and the excess paste at the smaller end sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care shall be taken not to compress the paste. The paste confined in the ring, resting on the plate, shall be placed under the rod, the larger end of which shall be brought in contact with the surface of the paste; the scale shall be then read, and the rod quickly released. The paste shall be of normal consistency when the rod settles to a point 10 mm. below the original surface in one-half minute after being released. The apparatus shall be free from all vibrations during the test. Trial pastes shall be made with varying percentages of water until the normal consistency is obtained. The amount of water required shall be expressed in percentage by weight of the dry cement.

41. The consistency of standard mortar shall depend on the amount of water required to produce a paste of normal consistency from the same sample of cement. Having determined the normal consistency of the sample, the consistency of standard mortar made from the same sample shall be as indicated in Table I, the values being in percentage of the combined dry weights of the cement and standard sand.

TABLE I.—PERCENTAGE OF WATER FOR STANDARD MORTARS

Percentage of Water for Neat Cement Paste of Normal Consistency.	Percentage of Water for One Cement, Three Standard Ottawa Sand.	Percentage of Water for Neat Cement Paste of Normal Consistency.	Percentage of Water for One Cement, Three Standard Ottawa Sand.
15	9.0	23	10.3
16	9.2	24	10.5
17	9.3	25	10.7
18	9.5	26	10.8
19	9.7	27	11.0
20	9.8	28	11.2
21	10.0	29	11.3
22	10.2	30	11.5

XII. DETERMINATION OF SOUNDNESS*

42. APPARATUS. A steam apparatus, which can be maintained at a temperature between 98 and 100° C., or one similar to that shown in Fig. 275, is recom-

* Unsoundness is usually manifested by change in volume which causes distortion, cracking, checking or disintegration.

Pats improperly made or exposed to drying may develop what are known as shrinkage cracks within the first twenty-four hours and are not an indication of unsoundness. These conditions are illustrated in Fig. 276.

The failure of the pats to remain on the glass or the cracking of the glass to which the pats are attached does not necessarily indicate unsoundness.

mended. The capacity of this apparatus may be increased by using a rack for holding the pats in a vertical or inclined position.

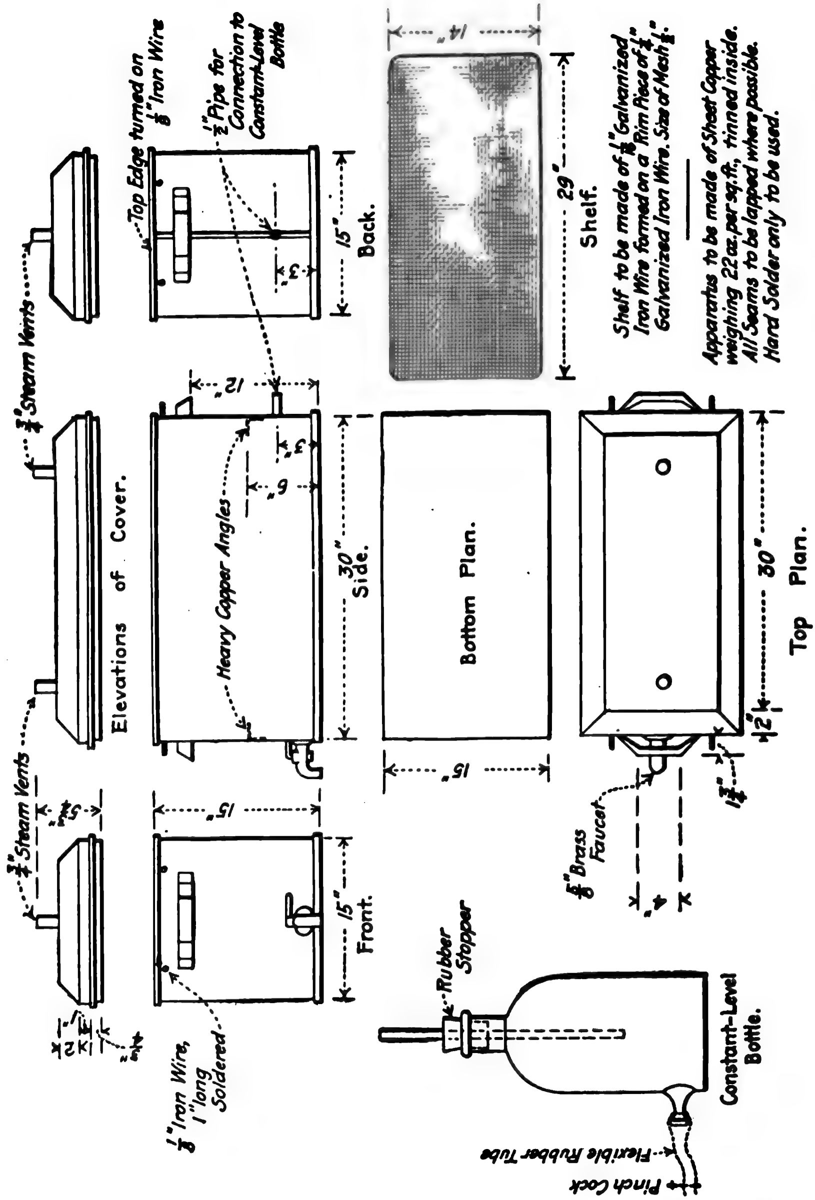


FIG. 275.—APPARATUS FOR MAKING SOUNDNESS TEST OF CEMENT.

43. METHOD. A pat from cement paste of normal consistency about 3 inches in diameter, ½ inch thick at the center, and tapering to a thin edge, shall be made

on clean glass plates about 4 inches square, and stored in moist air for twenty-four hours. In molding the pat, the cement paste shall first be flattened on the

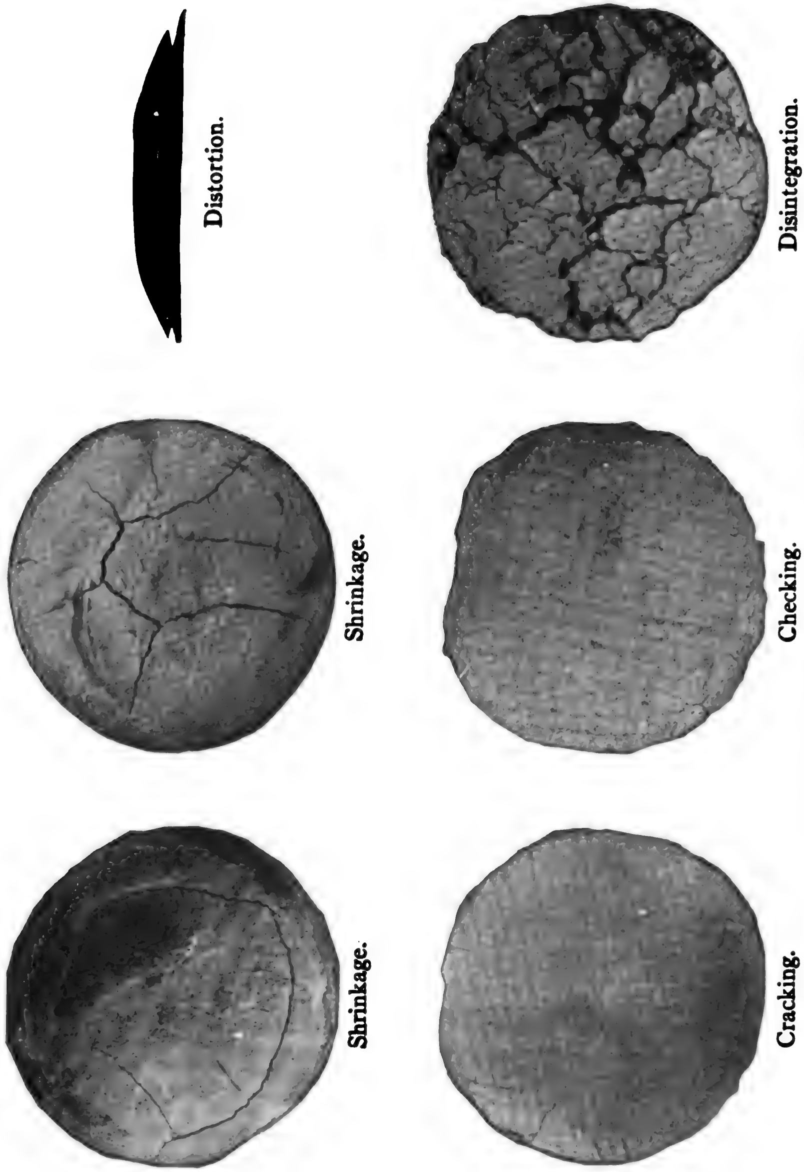


Fig. 276.—Typical Failures in Soundness Test.

glass and the pat then formed by drawing the trowel from the outer edge toward the center.

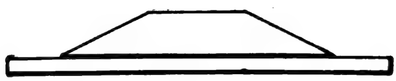
44. The pat shall then be placed in an atmosphere of steam at a temperature between 98 and 100° C. upon a suitable support 1 inch above boiling water for five hours.

45. Should the pat leave the plate, distortion may be detected best with a straight edge applied to the surface which was in contact with the plate.

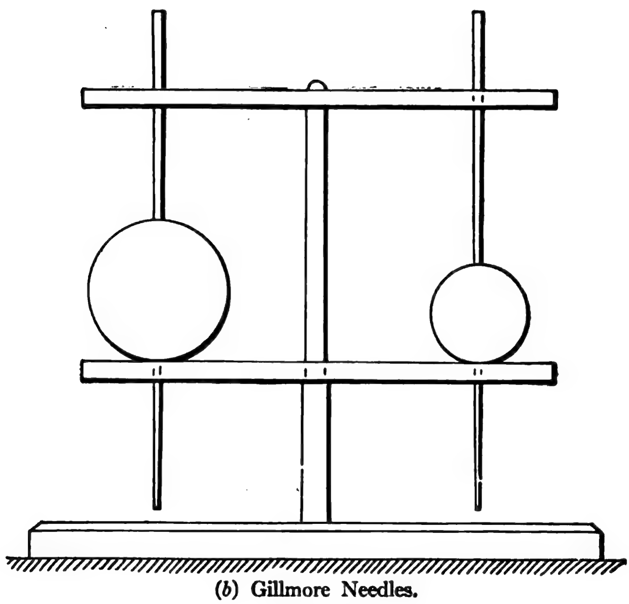
XIII. Determination of Time of Setting

46. The following are alternate methods, either of which may be used as ordered:

47. Vicat Apparatus. The time of setting shall be determined with the Vicat apparatus described in Section 39. (See Fig. 274.)



(*a*) Pat with Top Surface Flattened for Determining Time of Setting by Gillmore Method.



(*b*) Gillmore Needles.

Fig. 277.

48. Vicat Method. A paste of normal consistency shall be molded in the hard rubber ring *G* as described in Section 40, and placed under the rod *B*, the smaller end of which shall then be carefully brought in contact with the surface of the paste, and the rod quickly released. The initial set shall be said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate in one-half minute after being released; and the final set, when the needle does not sink visibly into the paste. The test pieces shall be kept in moist air during the test.

This may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth, kept from contact with them by means of a wire screen; or they may be stored in a moist closet. Care shall be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is therefore only approximate.

49. GILLMORE NEEDLES. The time of setting shall be determined by the Gillmore needles. The Gillmore needles should preferably be mounted as shown in Fig. 277 (*b*).

50. GILLMORE METHOD. The time of setting shall be determined as follows: A pat of neat cement paste about 3 inches in diameter and ½ inch in thickness with a flat top (Fig. 277 (*a*)), mixed to a normal consistency, shall be kept in moist air at the temperature maintained as nearly as practicable at 21° C. (70° F.). The cement shall be considered to have acquired its initial set when the pat will bear, without appreciable indentation, the Gillmore needle $\frac{1}{12}$ inch in diameter, loaded to weigh ¼ pound. The final set has been acquired when the pat will bear without appreciable indentation, the Gillmore needle $\frac{1}{24}$ inch in diameter, loaded to weigh 1 pound. In making the test, the needles shall be held in a vertical position, and applied lightly to the surface of the pat.

XIV. TENSION TESTS

51. FORM TEST PIECE. The form of test piece shown in Fig. 278 shall be used. The molds shall be made of non-corroding metal and have sufficient material in the sides to prevent spreading during molding. Gang molds when used shall be of the type shown in Fig. 279. Molds shall be wiped with an oily cloth before using.

52. STANDARD SAND. The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve. This sand may be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.

53. This sand, having passed the No. 20 sieve, shall be considered standard when not more than 5 grams pass the No. 30 sieve after one minute continuous sieving of a 500-gram sample.

54. The sieves shall conform to the following specifications:

The No. 20 sieve shall have between 19.5 and 20.5 wires per whole inch of the warp wires and between 19 and 21 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0165 inch and the average diameter shall not be outside the limits of 0.160 and 0.170 inch.

The No. 30 sieve shall have between 29.5 and 30.5 wires per whole inch of the warp wires and between 28.5 and 31.5 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0110 inch and the average diameter shall not be outside the limits 0.0105 to 0.0115 inch.

55. MOLDING. Immediately after mixing, the standard mortar shall be placed in the molds, pressed in firmly with the thumbs and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mold and smoothed off with a trowel; the trowel shall be drawn over the mold in such a manner as to exert a moderate pressure on the material. The mold shall then

be turned over and the operation of heaping, thumbing and smoothing off repeated.

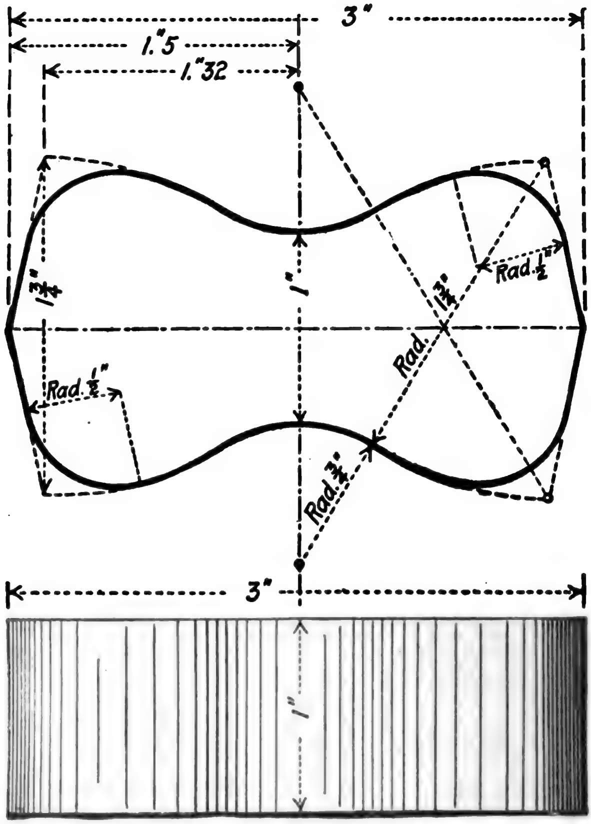


FIG. 278.—DETAILS FOR BRIQUETTE.

56. TESTING. Tests shall be made with any standard machine. The briquettes shall be tested as soon as they are removed from the water. The bearing

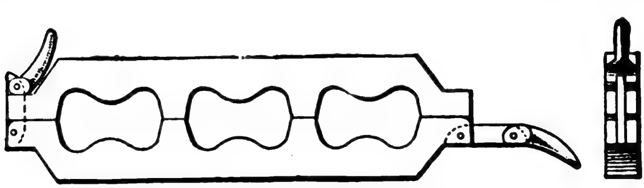


FIG. 279.—GANG MOLD.

surfaces of the clips and briquettes shall be free from grains of sand or dirt. The briquettes shall be carefully centered and the load applied continuously at the rate of 600 pounds per minute.

57. Testing machines should be frequently calibrated in order to determine their accuracy.

58. Faulty Briquettes. Briquettes that are manifestly faulty, or which give strengths differing more than 15 per cent from the average value of all test pieces made from the same sample and broken at the same period, shall not be considered in determining the tensile strength.

XV. Storage of Test Pieces

59. Apparatus. The moist closet may consist of a soapstone, slate or concrete box, or a wooden box lined with metal. If a wooden box is used, the interior should be covered with felt or broad wicking kept wet. The bottom of the moist closet should be covered with water. The interior of the closet should be provided with non-absorbent shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily.

60. Methods. Unless otherwise specified all test pieces, immediately after molding, shall be placed in the moist closet for from twenty to twenty-four hours.

61. The briquettes shall be kept in molds on glass plates in the moist closet for at least twenty hours. After twenty-four hours in moist air the briquettes shall be immersed in clean water in storage tanks of non-corroding material.

62. The air and water shall be maintained as nearly as practicable at a temperature of 21° C. (70° F.).

APPENDIX VI

METAL SHEET-PILING

THE prediction was made, in the first edition of this book, at the end of Chapter VI, that Metal Sheet-piling would doubtless come into use as timber became more expensive to use, and at the end of Chapter VIII mention is made of work at Cuxhaven Harbor, Germany, where Metal Sheet-piles were used. An account is also given of the Friestedt Patent Interlocking Sheet-piling, which is almost identical with Metal Sheet-piling described in Volume I of the "Transactions of the Institution of Civil Engineers." It is worthy of comment that this has probably been lost sight of by engineers, and even "The Engineer" in a review of the first edition of this book makes the statement:

"Numerous existing examples of coffer-dams constructed of sheet-piling are described and illustrated, and Mr. Fowler endorses a statement recently made in our columns by remarking that "the growing scarcity of timber will doubtless lead to the use of metal at some time in the future to replace sheet-piling for coffer-dams." So that the following paper on Metal Sheet-piling, published in 1836, will doubtless prove of great interest to engineers, if not of considerable value:

"Memoir on the use of Cast Iron in Piling, particularly at Brunswick Wharf, Blackwall. By Michael A. Borthwick, A. Inst. C.E."

A short sketch of the introduction and use of cast iron in piling may not be considered an inappropriate accompaniment to an account of one of the most recent works in which it has been adopted.

Public attention was first drawn to such an application of iron by Mr. Ewart, of Manchester, now of His Majesty's Dock-yard, at Woolwich; but though this merit is certainly due to that ingenious gentleman, he had been, as it afterwards proved, anticipated in the idea by the late Mr. Mathews, of Bridlington, who, previously to the date of Mr. Ewart's patent, had used cast-iron sheet-piles in the foundations of the head of the north pier of that harbor. These piles were of different forms; in the margin (Fig. 280) is given a cross-section of one of, I believe, the most common, in which it will be seen the adjoining piles dovetail to each other, while in others, I have been informed, they merely overlap. Their length was about 8 or 9 feet, their width from 21 inches to 2 feet, and their thickness half an inch.

In ignorance of Mr. Mathews' proceedings, Mr. Ewart, in the beginning of 1822, took out a patent for a new method of making coffer-dams, which he proposed to effect by employing plates of cast iron, held together by cramps fitted to dovetailed edges on the piles. A section of these piles, taken from some that have been used, is shown in the accompanying sketch (Fig. 281).

A detail of the mode in which it was proposed to combine them so as to form a coffer-dam might be out of place, in a paper that has reference more to the use of iron piling for permanent purposes; the plan, as described in the specification of the patent, is to be found in the Repertory of Arts, and an abstract of it in the *London Journal of Arts and Sciences* for the year 1822. The length of the piles is therein stated as intended to be from 10 to 15 feet, which is, I understand, about what they have generally been made, and for cases requiring a greater depth, a mode is described of lengthening the piles, by placing one above another, and securing the horizontal joints by means of dovetailed cramps.

Though, on being apprised of what had been done at Burlington, Mr. Ewart did not defend his patent, his piles have been pretty extensively adopted, particularly by Mr. Mylne, of New River Head, London, and Mr. Hartley, of Liverpool. Besides other operations in the important public work under his charge, the former gentlemen used the piles, soon after their invention, with complete success in a coffer-dam of considerable size, constructed in the river Thames for the purpose of putting in a suction-pipe opposite the New River Company's establishment at Broken Wharf. They have also been used with advantage by Mr. Hartley, in founding the pier heads of the basin of George's Dock, and various parts of the walls of some of the other docks at Liverpool, as also in putting in the foundations of the south river-wall.

FIG. 280.—MATTHEWS' CAST-IRON SHEET-PILE.

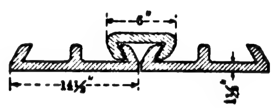


FIG. 281.—EWART'S CAST-IRON SHEET-PILE.

Looking at the dovetailed form of these piles, one would, I think, have been inclined to anticipate difficulty in driving them, but this does not seem to have been met with to any extent in practice, at least in coffer-dams, the original object of the invention. On this point I have pleasure in being able to quote some observations of Mr. John B. Hartley, which contain the results of the Liverpool experience: " Considerable care," he writes, " is required in keeping the piles in a vertical position, as they are apt to shrink every blow and drive slanting. They require to be driven between two heavy balks of timber to keep them in a straight line, as they expose very little section to the blow of the ram, and are so sharp that they are easily driven out of a right line. There is another very necessary precaution to be taken, which is the keeping of the fall in the same line as the pile; otherwise the ram descending on the pile and not striking it fairly, all parts equally, the chances are that, if in a pretty stiff stratum, the head breaks off in shivers, and the pile must be drawn, which is sometimes no easy matter." He concludes by saying, " these piles are on the whole the most useful tools you can use for their purpose (coffer-damming). I believe they have had as extensive a trial at the Liverpool Docks as anywhere else, and certainly with success. They have generally been driven with the ringing or hand engine and rams of 3 or 4 cwt., a front and back pile being driven at the same time by one ram."

In the work at Broken Wharf, the practice was to insert the piles and cramps

all round the dam first, and drive them a moderate distance into the ground, then to pass the engine repeatedly round and send them down gradually, instead of driving them home at once; and Mr. Mylne has mentioned to me that while this was in progress, the piles being at the time but slightly driven, he was somewhat alarmed one morning at finding that the run of the water had elevated one end of the dam considerably above the other. The dovetails, however, held good, and proper precautions being taken, the return of the tide put all right again without at all crippling the work, the movement having been regular all over the dam. I ought to add that these dams are still used in the works on the New River, four sets being generally kept in hand, and that the ringing engine is always employed, and the above stated method of driving followed.

I have perhaps dwelt longer on Mr. Ewart's project than I should otherwise have done, from a feeling that from his labors has sprung much that has followed in the way of iron piling; and besides, it may be observed, the remarks as to driving are not entirely limited in their application to this particular description of pile. The next work that occurs was executed by Mr. Walker in 1824; this was the rebuilding of the return end of the quay-wall of Downes Wharf, Saint Katherine's, which had been undermined by the wash from the Hermitage entrance of the London Docks. With a view to a more effectual resistance of a like action in future, iron instead of wood sheet-piling was introduced in the foundation of the wall in question; and though, if one may judge from the specification of the patent, no application of his plan of so permanent a nature seems to have been contemplated by Mr. Ewart, the work was begun according to it, but it was afterwards modified at the request of the contractor, so as to give the section of pile shown in the margin (Fig. 282), the flanch being in front or outside. Although, as has been already seen, the piles in their original form may be easily enough driven in some cases, it was found impossible to get them down in a regular line to the depth required in the present instance, through the hard material that had to be penetrated, and by which in fact they were surrounded and pressed for nearly their whole length of 14 feet.

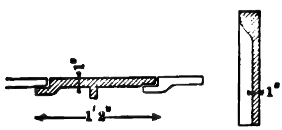


FIG. 282.—EWART'S MODIFIED SHEET-PILE.

A work on a much larger scale than any yet mentioned now presents itself, the wharfing at the sea entrance of the Norwich and Lowestoft navigation. In this Mr. Cubitt has adopted sheet-piling exclusively without the intervention of main or guide piles; the form and section will be seen by the accompanying sketches (Fig. 283), which it is almost unnecessary to observe are not drawn to the same scale, the transverse section being considerably enlarged beyond the other two. The piles are all 30 feet long; their weight is about a ton and a half each. The back flanch, which is shown at the deepest on the cross-section, tapers gradually to about 6 inches at top, where the angles are blocked in to form a head for driving, and is diminished at the lower end by steps or set-offs of parallel width with square ends, instead of a straight or curving line, as the latter shape was found to have a tendency to press the pile forward, whereas by the plan adopted it drove as fairly as if the flanch had been continued its full width to the foot of the pile. The driving was all effected by means of crab engines with monkeys about as heavy as the piles, no more fall being allowed than was necessary to send them

down, and the whole is secured by land ties, two in height, at intervals of six feet. The entire length of wharfing thus constructed is about 2000 feet.

From the form of the pile, according to this plan, giving so thin an abutting surface, and the joints not being covered in any way, close and accurate driving seems essential to its efficacy, and the nature of the ground (sand mixed with shingle) would have made this a somewhat troublesome operation at Lowestoft, but for the plan that was taken to insure precision. This consisted in riveting close to the lower end of the pile about to be driven a pair of strong wrought-iron cheeks projecting beyond the edge about 2 or 3 inches, which clasping the pile already driven, served as a guide or groove to keep the piles flush, however this the edge * and the tendency to turn out or in at the heel was counteracted after a few trials by giving a greater or less bevel to the front or back face. With these appliances the piling was pretty closely driven, and the work, which was completed in 1832, has been found fully to answer the object of supporting the sides of the cut from Lake Lothing to the sea against the effects of the very ingenious and powerful sluicing apparatus provided in the lock at that place.

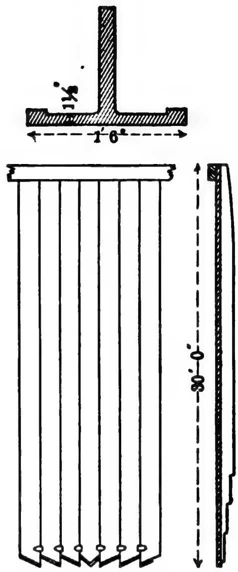


FIG. 283.—CUBITT'S IRON SHEET-PILING.

About a year later than the above, Mr. Sibley constructed an iron wharfing on the Lea Cut at Limehouse on an opposite principle, sheet-piling being in this case altogether discarded, and the work consisting of flat plates let down in grooves on the sides of guide-piles of an elliptical form according to the section opposite (Fig. 284), driven at distances of 10 feet. These piles are 20 feet long, weigh about 1½ tons each, and are 9 feet apart; they are hollow throughout, to enable a passage for them to be bored in the soil by means of an auger passed through them, and so ease the driving and are filled with concrete; each pile is land-tied, and the plates extend to within 6 feet of the point. A similar wharfing, but on a larger scale, has since been made on each side of the Thames, adjoining New London Bridge; that on the city side rather an extensive work, the piles in it being 43 feet long (cast in two unequal lengths with a spigot and faucet joint), of a cylindrical form, 12 inches diameter, and of metal 1½ inches thick, and each pile being secured by two tiers of ties of 2-inch square iron carried 70 or 80 feet back, to resist the great depth of filling up or backing.

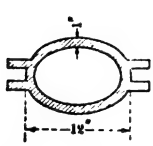


FIG. 284.—SIBLEY IRON SHEET-PILING.

The plan just described seems well enough adapted for situations where any great increase of depth is not likely to take place. The absolute depth is not so important, though where this is considerable, it may be questionable whether a heavy wharf would not be the better for the protection of a continuous row of piling at foot; the strong land-tying necessary in the last-mentioned work seems to point to this.

* This plan has, I believe, been followed by Mr. Cubitt in driving timber-piling also, in cases requiring nicety of work.

I now come to the quay-wall constructed in 1833-34 by Messrs. Walker and Burges on the river Thames, in front of the East India Docks at Blackwall and since named Brunswick Wharf. The object of this work was to afford accommodation for the largest class of steam-vessels at all times of tide, for which the old quay, even had it not been in a state of decay, was not adapted from the shallowness of the water in front of it. To effect this, the first idea was to run out two or three jetties from the wharf, but this was soon abandoned, and a new river-wall resolved on; and advantage was taken of the occasion to improve the line of frontage by an extension into the river, under the sanction of the Navigation Committee of the Port of London, varying from a point at the east end to about 25 feet at the other extremity. The use of iron in the work was, I have understood, suggested by Mr. Cotton, deputy chairman at the time, and for many years an active member of the most respectable and liberal body then in the direction of the East India Dock Company, and the adoption of the proposal was facilitated by the circumstance which probably led in the first instance to its being made, namely, the low price of the material at the period, the contract being little more than 7 pounds per ton delivered in the Thames.

In the accompanying drawing (Fig. 285) an attempt is made to show the mode of construction that was followed, so as to avoid the necessity for much written detail. The first operation was to dig a trench 2 yards deep in the intended line, and this was immediately followed by the driving of the timber guide-piles. The deepening in front, which, to give the required depth of 10 feet at low water, was as much as 12 feet, was not done until near the conclusion of the work; to have effected it in the first instance would without any countervailing advantage, except some saving in the driving, have been attended with the double expense of removing the ground forming the original bottom between the old and new lines of wharfing, and afterwards refilling the void so left by a material that would require time to make it of equal solidity; and even if this had been otherwise, such an attempt would have endangered the old wall, or rather would have been fatal to it. The permanent piling was next begun, the main piles being driven first at intervals of 7 feet, and the intermediate spaces or bays then filled in, working always from right to left, towards which the drafts of the sheet-piles were pointed. The ground is a coarse gravel, with a stratum of the hard Blackwall rock occurring in places, and some trouble was occasionally experienced from its tendency to turn the piles from the proper direction, but, due attention being paid to the form of the points, the driving was on the whole effected pretty regularly, but few of the bays requiring closing piles specially made for them, so that the work may be said to be nearly iron and iron from end to end; at the same time, the vertical joints of the piling being all covered, as will be noticed presently, any slight imperfection in this respect is no serious detriment to the work as a whole.

The main piles are in two pieces, the lower end of the upper one being formed so as to fit into a socket on the top of the under length, and the joining made good by means of a strong screw-bolt; the only object of this was to insure a supply of truer castings, and lessen the difficulty of transporting such unwieldy masses from Northumberland and Staffordshire to London.* Each sheet-pile is secured at the top by two bolts to the uppermost wale of the woodwork behind, and the edge of the end ones of each bay, it will be observed, passes behind the

* The Birtley Iron Company, Newcastle-on-Tyne, were the contractors for the ironwork but a portion was supplied by the Horsely Iron Company. Mr. M'Intosh, of Bloomsbury Square, had the contract for driving the piles and fixing the work.

adjoining main pile, while the other joints are overlapped by the bosses with which all the sheet-piles except the closers are furnished on one side. Besides adding to the perfection and security of the work by breaking the joints, so that

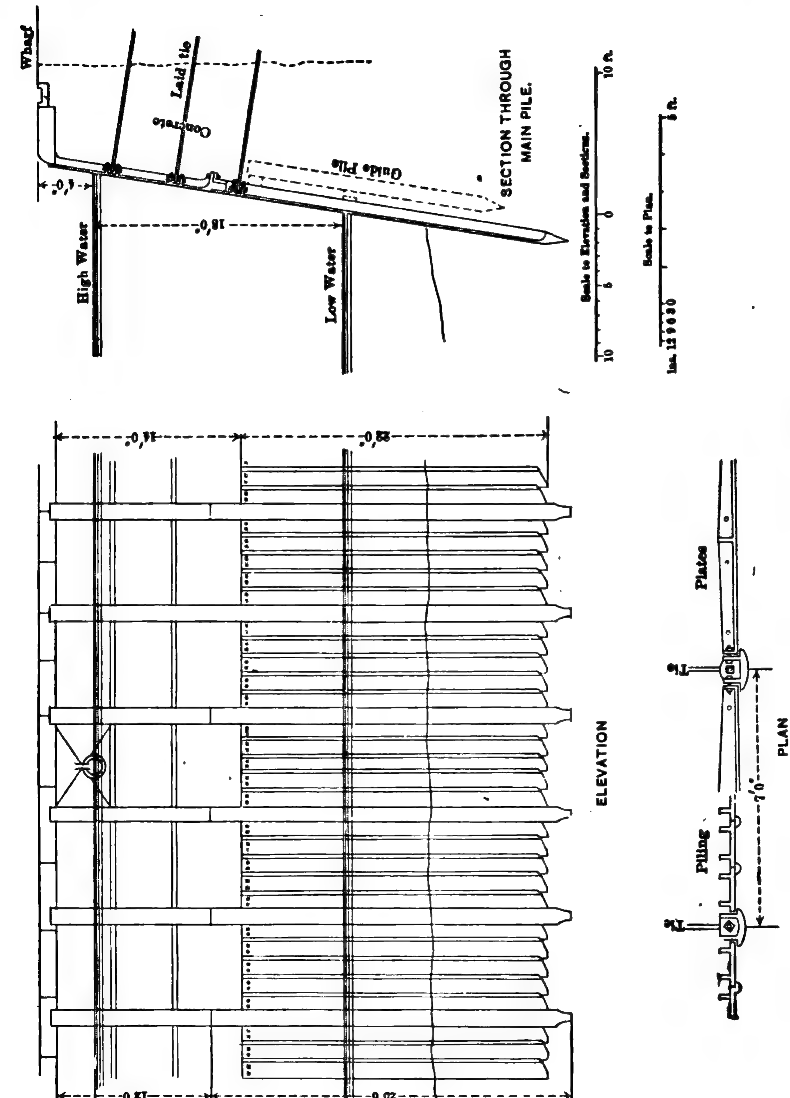


FIG. 285.—BRUNSWICK WHARF, IRON SHEET-PILING.

the water (if it penetrate, as with even the best pile-driving it will) cannot draw the backing from its place, these projections appear to me to relieve the appearance of the otherwise too uniform face; and a like effect is produced by the horizontal fillets on the lower edges of the plates above, which also mask the joints. These

plates, filling up the spaces over the sheet-piling, are bolted to the main piles and to each other in the manner shown, and the joints stopped with iron cement. Where the mooring-rings come, the plates are cast concave, with a hole perforated in the middle to allow a bolt to pass through, and this bolt is secured, as well as the land-ties from the main piles, to the old wharf, which was not otherwise disturbed, or to needle-piles driven adjoining it. The backing consists of a concrete of lime and gravel, in the proportion of about one to ten, extending down to the solid bottom. The coping with the water channel in its rear is of Devonshire granite; the water is conveyed from the channel at intervals by pipes, extending from gratings in the bottom in a slanting line to the lowermost plate, discharging themselves immediately above the sheet-piling.

The main piles were originally proposed to be hollow in section, according to the sketch opposite (Fig. 286); but this was given up on further consideration of the uncertainty of procuring sound castings of the intended form, and of the greater liability to break afterwards from a blow sidewise. The solid form shown on the plate was therefore adopted, according to which the lower lengths weighed about 28 cwt.; and that this was not too much was shown by the circumstance of several of the piles, particularly the early ones, breaking in the testing or driving, and showing in the fracture the danger of even a slight defect. The greater care subsequently taken at the foundry, and probably also greater experience in driving, made accidents of this kind of rarer occurrence in the later stages of the work; and it may be mentioned as no bad proof of the care of all parties, that of upwards of six hundred piles, including both descriptions, only sixteen broke in driving, seven being of one sort, and nine of the other; the failure was in five cases attributed to strains in driving, and to imperfections of casting in the other eleven. The sheet-piles, which bear a considerable resemblance in their general outline to those used at Downes Wharf ten years before, were proposed to be an inch thick, but it was found necessary to increase this dimension, and some of them were as much as 1¼ inches; the average, however, was not above 1⅛ inches, and the weight of each pile 17 cwt. The length of the wharf is about 720 feet, and the whole weight of iron used upwards of 900 tons.

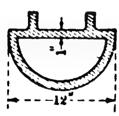


FIG. 286. — ORIGINAL PILE PROPOSED FOR BRUNSWICK.

The crab engine was employed invariably, the heads of the piles being covered with a slip of ¾-inch elm, to distribute the force of the blow equally over the iron, and prevent jarring. The monkeys used weighed from 13 to 15 cwt. each, and it was found necessary to limit the fall to a height of 3 feet 6 inches, and sometimes less, when the resistance proved more than usually great and the pile showed a tendency to turn from its straightforward course. The driving throughout was very hard, more especially at the west end, where the sheet-piles in four bays could not be forced to the full depth, the space above being in two of them made up with two plates in height, and in the other two admitting only one, instead of three as in the rest of the work. Driving was the only means resorted to, or indeed practicable in the gravelly soil that prevailed. Had the bottom been clay or other similar substance, the plan of boring to receive the points, that has been followed elsewhere, might probably have been partially adopted in the main piles with advantage; but I should say, certainly not to the extent of depending mainly upon it for getting the piles home to their places.

I cannot quit the subject of the Brunswick Wharf without stating that his avocations alone have prevented Mr. George Bidder's association with me in the account of a work, the execution of which he had, under Messrs. Walker and Burges, the charge of superintending. Though rejoicing at the cause, I cannot help regretting the circumstance in the present instance, as such co-operation on the part of my friend would, I feel, have given this paper an interest and a value it has now but little claim to. I take this opportunity also of acknowledging my obligation to several of the gentlemen above named in connection with the previous use of iron piling, whose kindness has enabled me to make the preliminary review much fuller than I had at one time any expectation of having the power to do.

It remains for me only, in conclusion, to advert to a consideration that ought not to be lost sight of in deciding upon the eligibility of cast-iron wharfing—I mean the action of water upon it. I do not recollect any observations made so as to enable a practical inference to be drawn from them; but the importance of the subject seems to claim attention, and possibly even this notice may be the means of inducing it from those who have the opportunity.

The investigation belongs perhaps rather to chemistry than engineering, but notwithstanding the practical turn some of the most distinguished cultivators of that science have given their researches, little I believe has yet been done to explain the present question. How iron is affected by water in its various states, and in what manner the action on wrought differs from that on cast iron, are interesting points, still, so far as my information goes, to be determined; and they are not likely, to be so in a satisfactory manner until some one competent to the task calls a series of well-conducted experiments in aid, as every day shows more clearly the uncertainty of analogical reasoning, however appararently strict, on such subjects. But whatever the modus operandi between cause and effect, that decomposition of the metal, more or less rapid, gradually goes on from the action of water, seems to admit of no doubt. Professor Faraday, in a letter to Captain Brown, says," Cast iron is certainly liable to great injury from constant immersion in salt water, and I think you would find few, if any exceptions, provided the water and the iron are in contact."* And the saline principle, to use a somewhat antiquated form of expression, though a great accelerator of the process, does not appear to be altogether an essential to it;† at least, I know a case that happened in a part of the river Thames where the water cannot be said to be more than brackish at any time, and indeed is generally quite fresh, in which cast iron, after being immersed for little more than twenty years, was, on being withdrawn from the water, found so soft as to yield to the penknife; and the original surface of the iron referred to—it was the socket-plate to the heel-post of a lock-gate—had not been submitted to the tool, in which case it is well known the water would have operated with much greater effect.

But though I have thought it well to glance at the above case occurring in water, always except on rare occasions fresh, the sea is no doubt in practice, the invader whose inroads are most alarming. Instances might easily be cited in proof of the ravages committed by that active enemy, though not perhaps noted so circumstantially as is desirable, but I am unwilling to lengthen this communication further, and shall therefore confine myself to a passing allusion

* Description of a Bronze or Cast-iron Columnal Lighthouse, etc., by Capt. Brown, R.N.

† The difference between sea and other water, in operating with the galvanic battery, is much less considerable than that between the latter and distilled, but it is between salt and fresh that the practical question lies in the present case.

to the example on a large scale, and after long trial, furnished by the state of the guns taken from the wreck of the *Royal George*, as described at a late meeting of the Institution;* and to a similar instance mentioned by Berzelius, in a passage which I quote at length, not so much however in confirmation of so well established a fact as the eventual decomposition of cast iron by the action of water, as for the properties mentioned of the substance into which the metal is resolved. The extract is as follows:

"Quand la fonte reste long-temps sous l'eau, elle est decomposée; l'acide carbonique contenu dans l'eau dissout le fer et l'entraine; il reste une masse grise qui ressemble à la plombagine. Lorsqu'on retira de l'eau, il y a quelques annés, les canons d'un vaisseau qui avait coulé à fond cinquante ans auparavant, aux environs de Carlscrona, on les trouva au tiers converti en une pareille masse poreuse; à peine étaient-ils à l'air depuis un quart d'heure, qu'ils commencèrent à s'échauffer tellement, que l'eau qui y restait encore s'échappa sous forme de vapeur, et qu'il fut impossible d'y toucher. Depuis, Macculloch a observé † que le corps analogue à la plombagine qui se forme ainsi présente toujours ce phénomène, et que ce corps s'échauffe presque jusqu'au rouge, en absorbant de l'oxygène. Ou ne sait pas précisement ce qui se passe dans ce cas."—Traité de Chimie, Tom, III, p. 273.

* Min. of Convers. Vol. V., No. 12.

† The observation referred to by Berzelius in the above occurs in Macculloch's Western Isles of Scotland (I think in the account of the island of Mull), where an explanation of the phenomenon was first attempted, though, if on such a subject I may "hint a doubt," not to my mind quite a satisfactory one. A more perfect solution will probably be furnished by whoever, availing himself of the powerful means of chemical analysis now possessed, may undertake such an investigation of the whole question of the action of water on iron as I have ventured to allude to in the text.

APPENDIX VII

SPECIFICATIONS FOR FLOATING PILE DRIVER U. S. ENGINEERS, COLUMBIA RIVER, OREGON

DETAILED SPECIFICATIONS

General Description.—The work to be done under these specifications includes the building of a scow barge; erecting thereon a set of gins and a house; and furnishing and installing all the machinery hereinafter specified. All patterns required for the hammer, sheaves, bearings, etc., shall become the property of the United States and shall be delivered with the driver.

Material and Workmanship.—All lumber shall be of the best quality close-grain yellow fir, except where otherwise specified, without shakes, rot splits, unsound or large knots, sap or other imperfections. Decking and planking shall be edge grain, free from knots on face and edges, thoroughly seasoned and dry, but not kiln dried. All other timber shall be as well seasoned as time and circumstances will permit. All lumber shall be surfaced on four sides, the sizes given being for the timber in the rough.

All fastenings shall be galvanized. All bolts shall have washers under the nuts. Holes for fastenings shall be bored, allowing $\frac{1}{16}''$ for drifting.

All valves shall be "Lunkenheimer," "Powell," or equal; those $3''$ and under shall be all brass and those above $3''$ shall be cast iron with brass mountings.

All other materials shall be the best of their respective kinds and all work shall be done in a workmanlike manner.

Dimensions.—Length over all, 70′; width, 24′; moulded depth, 4′; crown of deck, 4″; height of gins, 66′.

Gunwales.—Each gunwale shall be built up of 5 strakes of the following dimensions: the bottom strake, $12''\times 12''$ in one length; the second strake, $6''\times 12''$ in two lengths; the third strake, $6''\times 12''$ in three lengths; the fourth strake, $6''\times 12''$ in two lengths, and the fifth strake, $6''\times 6''$ in one length. All joints shall be 4′ scarfed joints, well broken so that no joint is directly over another.

The gunwale timbers shall be well fitted and fastened to a $6''\times 6''$ rake timber and an 8″ knee as shown in Fig. 44. At the stern they shall be dovetailed into the transom as shown.

The gunwale strakes shall be fastened together with $\frac{3}{4}''$ drift-bolts and clinch-bolts, driven with $\frac{1}{16}''$ drift. A complete set of drift-bolts shall be driven for each strake as it is put up. They shall be spaced 2′ centers and each bolt shall go through two and one-half timbers, except the bottom set, which shall go through the second strake and within one inch of bottom of bottom strake. The clinch

bolts shall be spaced 8′ centers and extend through all strakes and be clinched over rings.

The strakes shall be closely joined together. The sides, scarfs and joints shall have calking seams $\frac{3}{16}''\times 3''$.

TRANSOMS.—The forward transom shall be built up of three $8''\times 12''$ timbers properly shaped for the crown of the deck and rake of plank.

The after transom shall be built up of 5 strakes of the following dimensions: the bottom strake, $12''\times 12''$; the next three, $6''\times 12$,″ and the fifth, $6''\times 10''$. They shall be properly shaped for the crown of deck and rabbeted for planking. All transom timbers shall be fitted and fastened the same as specified for gunwales. At each dovetailed corner there shall be one $\frac{3}{4}''$ bolt clinched over rings.

BULKHEADS.—There shall be two longitudinal bulkheads, built up of one $8''\times 8''$ strake in one length, one $6''\times 10''$ strake in six lengths, two $6''\times 10''$ strakes in three lengths with scarfed joints, and one $6''\times 10''$ strake in one length, all drift-bolted and through fastened the same as specified for gunwales. The bulkheads shall be secured to the after transom by $8''\times 8''$ posts and to the forward transom by 8″ knees as shown, all through bolted as directed.

TRUSSED STRINGERS.—There shall be five trussed stringers built up as shown in Fig. 47, with the moulded depth to suit the crown of deck. The bottom member shall be $8''\times 8''$ in one length; the top member, $6''\times 6''$ with joints as shown. The posts shall be $6''\times 6''$ and the diagonals $3''\times 4''$, all well fitted with close joints. At each end of each stringer there shall be one 8″ knee connection to the transoms fastened with seven $\frac{3}{4}''$ through bolts. At every other post there shall be one $\frac{3}{4}''$ bolt passing through both members and clinched over rings. The other fastenings shall be $\frac{1}{2}''$ and $\frac{3}{4}''$ bolts, placed as directed. Limbers $1\frac{1}{2}''\times 3''$ shall be cut as directed.

CLAMP STRINGERS.—The clamp stringers shall be $4''\times 6''$ secured to the gunwales by $\frac{5}{8}''$ carriage bolts spaced one foot apart, heads outside let into counterbored and plugged holes.

CROSS BEAMS.—There shall be five $8''\times 8''$ cross beams placed as shown with an 8″ knee at each end of each beam. The posts and diagonal braces shall be as shown, well fitted and fastened. There shall be seven $\frac{3}{4}''$ through bolts through each knee placed as directed, and a $\frac{3}{4}''$ drift-bolt at every crossing of beam and bottom stringers.

DECK BEAMS.—There shall be five $8''\times 6''$ deck beams; the others shall be $4''\times 6''$ spaced 2′ centers. The 8″ beams shall be fastened at each trussed stringer by two $\frac{5}{8}''$ carriage bolts, and at each end and at each bulkhead by two $\frac{3}{4}''$ drift-bolts. The 4″ beams shall be fastened at each end and at each trussed stringer by one $\frac{5}{8}''$ carriage bolt and at each bulkhead by one $\frac{5}{8}''$ drift-bolt.

STANCHIONS.—There shall be four $8''\times 8''$ stanchions connecting the after transom to the gunwales and bulkheads. There shall be six $8''\times 8''$ stanchions or tow posts arranged as shown in Figs. 44 and 45, with the corners of the upper part neatly mitered. All stanchions shall be securely bolted as directed with $\frac{3}{4}''$ carriage bolts, heads outside let into counterbored and plugged holes.

BOTTOM PLANKING.—The bottom planking shall be $4''\times 10''$ run athwartship as shown. The edges shall be slightly beveled to give sufficient calking seams, and the planks laid with close joints inside. Each plank shall be fastened at each crossing by three 8″ ship spikes. The heads shall be let into counterbores and plugged with wooden plugs dipped in white lead.

DECK PLANKING.—The deck planking shall be $3\frac{1}{2}''\times 6''$ in lengths of not

less than 32′ with butts well shifted; no two butts shall be on the same beam unless at least three planks intervene. Each plank shall be spiked at each crossing by two 7″ ship spikes; the heads shall be let into counterbored and plugged holes.

GUARDS.—The upper guard shall be 3″×12″ on sides and 3″×16″ across the ends; the lower guard, 3″×8″ continued along the bow and joined to the upper guard. At the after end, 3″×8″ vertical guards shall join the lower to the upper guard. All guards shall be well spiked with 8″ ship spikes and the heads shall be let into counterbored and plugged holes. ⅜″×6″×6′ corner irons shall be placed at each corner, both top and bottom, and fastened with eight 8″ countersunk spikes.

HATCHES.—There shall be three 2′×3′ hatches at each end, located as directed. Each shall be provided with both a removable lattice cover and a solid cover, the latter being flush with deck and provided with iron lifting rings.

CALKING.—All seams in bottom, gunwales, transoms, and deck shall be calked with three threads of best oakum, each thread well driven. All seams below the water line shall be payed with white lead and filled with a good grade of Portland cement troweled down smooth. The deck and all other seams above water line shall be payed over oakum with a good grade of pitch.

HOUSE.—A house 16′×30′6″ shall be built substantially as shown on the drawings. The coaming shall be 6″×6″ shaped to suit deck, the studding 4″×4″ spaced about 3′ centers or to conform to doors and windows, and mortised into coaming. The plate shall be 4″×6″ and the nailing strips, 2″×4″. All timbers shall be well framed and nailed as directed. There shall be one ¾″ tie-rod at each corner and two on each side, running through the carlins and deck beams or filling timbers. The carlins shall be 3″×12″ spaced 3′ centers and sawed with a 6″ camber. They shall be bolted at each end into the plate by a ½″ carriage bolt.

The carlins shall be covered with 1¼″×6″ T. & G. material, nailed at every crossing, over which shall be laid athwartship, in paint, No. 5, 22″ cotton duck. Laps shall be fastened with double pointed galvanized tacks driven diagonally. A water course shall be run as shown, leading water into lead and galvanized iron scupper pipes one located at each corner. The siding shall be 1¼″×6″ T. &.G material run vertically and nailed at every crossing with 8d. wire nails.

The after end shall be provided with a 16-oz. canvas hood, hung by brass hooks and eyes, made so as to fasten down and cover all parts of the engine outside of the house.

The doors shall be constructed as shown, the side and forward end doors hung on Richards No. 28, or equal, hangers and track, and the after end doors hung on Richards No. 235, size 1, or equal, swivel trolley hangers and track. The doors shall be fitted with heavy hasps and staples arranged for padlocks. All windows shall have 1⅜″ sash glazed with 26-oz. crystal glass. They shall be fitted to drop into ceiled pockets in the usual manner.

A work bench shall be constructed in a substantial manner of the dimensions shown. It shall have drawers and doors arranged as directed. A suitable tool board shall be installed on the wall near the bench.

GIN SILLS.—There shall be two longitudinal gin sills each 12″×12″×50′ long with drift-bolts every two feet extending at least 16″ into solid bulkheads. In addition there shall be two ¾″ bolts at each end and two at the back brace connections with nuts in pockets in the bulkheads. The machinery founda-

tion bolts shall be arranged in the same manner. There shall be one cross sill 12″×12″ in three lengths, fitted into longitudinal sills and corner stanchions, well drift-bolted and through bolted into transom, stanchions, and sills by ¾″ bolts.

GINS.—The gin timbers shall be 8½″×12″ by approximately 66′ long, in one length. Each shall be notched into the sill timber as shown, and bolted by five ¾″ carriage bolts at the bottom and at the top as shown in detail in Fig. 44. Each gin shall be beveled to fit and be faced with an 8″×11.25 lb. channel iron, fastened with ½″ countersunk head-bolts spaced 1′ centers staggered. The lower part of channels shall be in one length at least 55′ long, the splice to the upper part being made with a ⅜″×7″ plate, well riveted.

BACK BRACES.—There shall be two back braces each 5″×12″. Each brace shall have the lower end through bolted into sill timber by five ¾″ bolts and the upper end framed and fastened as shown in detail in Fig. 44. The two braces shall be joined together by 2″×6″ material, spaced 15″ centers, forming a ladder. Each piece shall be fastened by four 4″×¼″ ship spikes.

SIDE BRACES.—There shall be two side braces each 8″×12″ framed at top and bottom as shown, and bolted at each end by five ¾″ carriage bolts. They shall be joined together and to the loft timbers by six timbers shaped as shown. These timbers shall be bolted at each end and crossing by two ⅝″ carriage bolts.

LOFT TIMBERS.—There shall be fourteen 4″×10″ and twelve 4″×6″ loft timbers arranged as shown. They shall be well fitted into gins, back braces, and side braces and fastened with two ⅝″ carriage bolts at each end. In addition each pair of 4″×10″ timbers shall be tied together at the gin ends by a ¾″ rod with nuts on each end. A choking device with the necessary sheaves shall be fitted at the top loft.

DECKING.—Each loft shall be completely decked over, except for a space for hose sheave and counterweight, with 1¼″×8″ material laid with one-inch spaces, well nailed with 16d. wire nails. The space between longitudinal sills from gins to boiler and a working platform around engine shall be decked with 2″ material properly supported and fastened with 20d. wire nails.

DIAGONAL BRACES.—The diagonal braces shall be 4″×10″ and 6″×8″ arranged as shown. They shall be well fitted to gins and braces and secured at each end by two ⅝″ carriage bolts.

HEAD BLOCK.—The arrangement of the head block timbers, sheaves and bearings shall be as shown in detail in Fig. 48. The timbers shall be well fitted and securely bolted. The sheaves shall be cast steel with turned groove and pins. The pile sheave pin shall have a hole drilled in each end and connect with each sheave bearing, and be fitted with two compression grease cups, Lukenheimer Ideal No. 3, or equal. This sheave pin shall be held in place by a taper pin in each bearing. The hammer line sheave shall have the pin pressed in and pinned. The bearings shall be lined with genuine babbitt, bored true, channeled for oil and each shall be fitted with an automatic grease cup, Lukenheimer Ideal No. 3, or equal.

HAMMER AND ROPE.—The hammer shall be cast iron, weighing 3800 pounds, and shall be in accordance with detail drawing furnished. The hammer line and pile lines will be furnished by the United States.

HOSE SHEAVE AND FITTINGS.—The hose sheave, holder, guides, etc., shall be made complete as shown in detail and assembled as shown. The sheaves shall be cast iron and have turned grooves and pins, all fitted up in first-class

manner. Oil holes shall be provided where required. The cast-iron counterweight shall be made in sections and shall be at least 100 pounds heavier than the assembled sheave holder, sheave, and hose full of water. It shall connect to the sheave holder by a $\frac{1}{2}''$ pliable steel cable. $65'$ of $2\frac{1}{2}''$ double-jacket rubber-lined fire hose shall be furnished and connected to the water piping and both ends shall have hose couplings.

Oil Tanks.—There shall be two oil tanks, each $2'\ 8''$ in diameter by $16'$ long, built of $\frac{3}{16}''$ tank steel with bumped heads. They shall be tested for tightness by a hydrostatic pressure of 20 pounds. They shall be supported in saddles as shown and be held down by lugs, and screw bolts. Each tank shall have pressed steel flanges for filling pipe, sounding pipe, suction pipe, and drain plug.

Air Receiver.—The air receiver shall be $2'\ 6''$ in diameter by $10'$ long, supported in saddles and held down by lugs and screw bolts in the location shown. The shell shall be $\frac{5}{16}''$ flange steel and the bumped heads $\frac{3}{8}''$, well riveted as required. It shall be tested by a hydrostatic pressure of 225 pounds per square inch. Pressed steel flanges shall be provided for inlet, outlet and drain plug.

Engine.—The engine shall be an American Hoist & Derrick Company, or equal, $8\frac{1}{4}'' \times 10''$ double-cylinder engine with two drums and four clutch winch heads. It shall be built for a working steam pressure of 125 pounds per square inch. Each drum shall be $14''$ diameter and $27''$ long between flanges. The upper drum shall be lagged to a diameter of $18''$ for the hammer line. The width between foundation bolts shall be approximately $4'$. The engine shall be complete with throttle valve, lubricator, and efficient means for lubricating all bearings.

Steam Capstan.—A single-barrel steam capstan, American Ship Windlass Company, or equal, shall be installed in the location shown. Each steam cylinder shall be $5'' \times 7''$ and the barrel $10\frac{1}{2}''$ in diameter. Additional timbers shall be installed as found necessary in order to securely fasten it in place.

Boiler.—The boiler shall be 40 h.p., portable locomotive type, with steam dome, water bottom and water front. It shall be built for a working pressure of 125 pounds per square inch and subjected to the Hartford Boiler Insurance Company inspection under hydrostatic pressure of 190 pounds. The boiler shall be about $16'$ long and $42''$ diameter of shell. The design and make shall be approved by the contracting officer before installation.

The boiler shall be equipped with the following fittings: smoke stack built of 16 gage steel, with steel hood and housing at upper deck and four guy wires; two sets of grate bars for burning wood; a Glafke, or equal, oil burner with heater; a $1''$ Metropolitan, or equal, automatic injector; Crosby pop safety valve; $3''$ chime bell whistle, 3 gage cocks into shell; water column and gage; steam gage with syphon and cock; 2 feet valves; 2 feed check valves; 1 blowoff cock.

Pumps.—There shall be a $3'' \times 2'' \times 3''$ duplex-boiler feed pump Worthington, or equal. The pump shall have Tobin bronze piston rods, composition-lined cylinders and composition valves. It shall be properly supported as directed and be connected as specified under piping.

The jet pump shall be a $10'' \times 6'' \times 10''$ Worthington, or equal, outside center-packed plunger pump having Tobin bronze piston rods, and composition plungers with composition bushed plunger stuffing boxes. The steam pipe shall be fitted with a suitable sight-feed lubricator of approved design and the discharge shall have a $6''$ dial pressure gage. This pump shall be supported on $12'' \times 12''$ timbers and be securely fastened to them. The pipe connections shall be as specified under piping.

Air Compressor.—A Westinghouse, or equal, standard 11" steam-driven, water-cooled, air compressor shall be installed in the location shown, supported by two 4"×6" timbers framed into the house coaming and plate. The compressor shall be complete with governor, 4" pressure gage, and sight-feed lubricator.

A 50-gallon galvanized-iron cooling tank shall be installed near the compressor and connected to it and to the feed pump discharge line.

Air Piping.—The air compressor shall be connected to the receiver by 1¼" pipe with a 1¼" globe valve and a pop safety valve in a convenient location. A 1¼" globe valve shall be placed at the receiver outlet from which a 1¼" pipe shall lead to the back braces where it shall pass through the deck and connect with two ¾" globe valves and a 1" pipe. This pipe shall lead to the fourth loft, with a plugged "T" at each loft, and terminate in two ¾" globe valves.

Oil Piping.—Each oil tank shall have a 1½" combined sounding and vent pipe placed as directed. They shall have 2½" filling pipes leading into a "T," connecting with a 3" filling stand in the deck, which shall have a composition cover and hose connection. Each tank shall have a 1½" suction pipe leading in from the top and terminating in a 1½" gate valve near the boiler; from these valves connection shall be made to the burner.

Water Piping.—The sea chest will be of steel pipe, furnished by the United States. It shall be secured in place by eight countersunk head bolts through 8" filling timbers, placed as directed, and the hull shall be made water-tight around it. All water piping shall be galvanized.

The jet-pump suction shall have four separate suctions, made independent by using three 5" flanged cross valves, one 5" screwed angle valve and one 5" flanged "T," made up with 5" pipe as directed. One suction shall have a 5" foot valve and strainer and shall lead into the sea chest. The other three shall lead one into each bilge compartment. The discharge shall be in two branches, a 4" branch with angle valve leading into sea chest, and a 3" branch leading aft as shown, with two 3" gate valves placed as directed. The valves, fittings, etc., in the discharge pipe line shall be suitable for a working pressure of 175 pounds per square inch.

The feed pump and injector shall each have a 1½" suction, with foot valve and strainer, from the sea chest and an independent discharge through feed heater to boiler. A Harrisburg No. 10, or equal, copper-coil feed-water heater shall be installed in the location shown. It shall be connected so as to use exhaust steam for heating. The drain shall be led overboard.

Steam Piping.—The steam pipe to the engine shall be 2½" standard black pipe with a 2½" globe valve near the boiler. The pipe to the jet pump shall be 2" with a 2" globe valve near the pump. The feed pump shall have a ½" globe valve and be connected to the same pipe as the injector. A continuation of this pipe shall lead to the steam capstan which shall have a globe valve with stem extended through the deck in a convenient location. Three bilge syphons shall be installed, one in each compartment aft, discharging overboard.

The engine exhaust shall be 3" black iron pipe from the engine to a 3"×4"×3" "T" connecting with the jet-jump exhaust, which shall also be 3" pipe, and shall be led under false deck between gin sills. From the "T" a 4" black iron pipe shall lead to the feed-water heater and from the heater it shall join the safety valve exhaust and project 3' above upper deck. It shall be properly flashed where passing through upper deck. The feed pump and steam capstan exhausts shall lead into the 4" pipe before it enters the heater. All steam piping shall

be tested by a hydrostatic pressure of 190 pounds per square inch. Where practicable long-radius fittings shall be used in steam piping.

After the boiler has been painted it shall be covered with asbestos and magnesia bricks 1½″ thick wired in place and crevices smoothly troweled with plastic magnesia covering. The surface shall be smoothly glazed and canvased in an approved manner. After testing, steam pipes, steam cylinders, and heater shall be covered with sectional magnesia pipe covering jacketed with canvas and secured with brass bands.

CLEATS.—There shall be four 42″ cast-iron cleats located as directed. Each cleat shall be set on a 12″×36″×½″ steel plate, and shall be secured by two 1″ bolts passing through plate, deck, and an 8″ channel-iron bearing under two deck beams.

CHOCKS.—There shall be four roller chocks located as shown. Each sheave shall be 6″ diameter held between two plates, the lower one 12″ wide with outboard edge turned over edge of timber; the top plate shall be 8″ wide. Both plates shall be well bolted together and into sills and transoms.

GYPSEYS.—There shall be one ratchet gypsey windlass, "Providence" size C, or equal, well secured in the location shown. The length of shaft shall be as shown. There shall be two "Providence" size C, or equal, ratchet gypsey half windlasses, well fastened in the locations shown.

SALT POCKETS AND SALT.—Salt pockets shall be constructed on the inside of each gunwale by nailing 1″×8″ material to the clamp stringers between the deck beams. Pockets shall be formed on each side of each solid bulkhead by beveling the edges of 2″×8″ material so that it stands at an angle of about 30 degrees from the bulkhead, being nailed on the top edge to the under side of deck beams and on the bottom edge to the bulkhead. All salt pockets shall be filled with half-ground rock salt.

PAINTING.—All joints in gunwales, bulkheads and transoms shall be coated with boiled linseed oil applied at or about boiling temperature just before they are made up. All timber ends shall receive a coat of boiled linseed oil at boiling temperature before being covered, the posts and cross bracing ends in trussed stringers being dipped in boiling oil.

The entire bottom, sides and ends up to a 22″ water line shall be painted with two coats of Woolsey's or equal, copper paint, the last coat applied just before launching. The entire hull, except deck, above the water line, house, etc., shall receive three coats of pure white lead and boiled linseed oil, the last coat tinted to a light lead color.

The boiler, unfinished parts of engine, pumps, steam capstan, piping and all other iron work shall receive two coats of red lead and boiled linseed oil, the second coat tinted as directed. The smoke stack shall receive two coats of asphaltum paint.

APPENDIX VIII

UNITED STATES GOVERNMENT SPECIFICATIONS 59C1 FOR PORTLAND CEMENT*

Definition

(1) The cement shall be the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate mixture of properly proportioned argillaceous and calcareous substances, with only such additions subsequent to calcination as may be necessary to control certain properties. Such additions shall not exceed 3 per cent by weight of the calcined product.

Composition

(2) In the finished cement, the following limits shall not be exceeded:

	Per Cent.
Loss on ignition for 15 minutes	4
Insoluble residue	1
Sulphuric anhydride (SO_3)	1.75
Magnesia (MgO)	4

Specific Gravity

(3) The specific gravity of the cement shall be not less than 3.10. Should the cement as received fall below this requirement, a second test may be made upon a sample heated for thirty minutes at a very dull red heat.

Fineness

(4) Ninety-two per cent. of the cement, by weight, shall pass through the No. 100 sieve, and 75 per cent. shall pass through the No. 200 sieve.

Soundness

(5) Pats of neat cement prepared and treated as hereinafter prescribed shall remain firm and hard, and show no sign of distortion, checking, cracking, or disintegrating. If the cement fails to meet the prescribed steaming test, the

* (Prepared by departmental conference February 13, 1912; adopted by the Navy Department March 23, 1912.) Issued for the use of the naval establishment April 15, 1912. Superseding "Specifications 7C2" and "59C1" issued November 22, 1907, and November 10, 1909.

cement may be rejected or the steaming test repeated after 7 or more days, at the option of the engineer.

Time of Setting

(6) The cement shall not acquire its initial set in less than forty-five minutes, and must have aquired its final set within ten hours.

Tensile Stength

(7) Briquettes made of neat cement, after being kept in moist air for twenty-four hours and the rest of the time in water, shall develop tensile strength per square inch as follows:

	Pounds.
After 7 days	500
After 28 days	600

(8) Briquettes made of 1 part cement and 3 parts standard Ottawa sand, by weight, shall develop tensile strength per square inch as follows:

	Pounds.
After 7 days	200
After 28 days	275

(9) The average of the tensile strengths developed at each age by the briquettes in any set made from one sample is to be considered the strength of the sample at that age, excluding any results that are manifestly faulty.

(10) The average strength of the sand mortar briquettes at twenty-eight days shall show an increase over the average strength at seven days.

Brand

(11) Bids for furnishing cement or for doing work in which cement is to be used shall state the brand of cement proposed to be furnished and the mill at which made. The right is reserved to reject any cement which has not established itself as a high-grade Portland cement, and has not been made by the same mill for two years and given satisfaction in use for at least one year under climatic and other conditions at least equal in severity to those of the work proposed.

Packages

(12) The cement shall be delivered in sacks, barrels, or other suitable packages (to be specified by the engineer), and shall be dry and free from lumps. Each package shall be plainly labeled with the name of the brand and of the manufacturer.

(13) A sack of cement shall contain 94 pounds, net. A barrel shall contain 376 pounds, net. Any package that is short weight or broken or that contains damaged cement may be rejected, or accepted as a fractional package at the option of the engineer.

Inspection

(14) The cement shall be tested in accordance with the standard methods hereinafter prescribed. In general the cement will be inspected and tested after

delivery, but partial or complete inspection at the mill may be called for in the specifications or contract. Tests may be made to determine the chemical composition, specific gravity, fineness, soundness, time of setting, and tensile strength, and a cement may be rejected in case it fails to meet any of the specified requirements. An agent of the contractor may be present at the making of the tests or they may be repeated in his presence.

(15) In case of the failure of any of the tests, and if the contractor so desires, the engineer may, if he deems it to the interest of the United States, have any or all of the tests made or repeated by the Bureau of Standards, United States Department of Commerce and Labor, in the manner hereinafter specified, all expenses of such tests to be paid by the contractor. All such tests shall be made on samples furnished by the engineer.

Standard Methods of Testing—Sampling

(16) The selection of the samples for testing will be left to the engineer. The number of packages sampled and the quantity to be taken from each package will depend on the importance of the work, the number of tests to be made, and the facilities for making them.

(17) The samples should be so taken as to represent fairly the material, and, where conditions permit, at least 1 barrel in every 50 should be sampled. Before tests are made, samples shall be passed through a sieve having 20 meshes per linear inch to remove foreign material. Samples shall be tested separately for physical qualities, but for chemical analysis mixed samples may be used. Every sample should be tested for soundness, but the number of tests for other qualities will be left to the discretion of the engineer.

Chemical Analysis

(18) The method to be followed for the analysis of cement shall be that proposed by the committee on uniformity in the analysis of materials for the Portland cement industry, reported in the *Journal of the Society for Chemical Industry* (Vol. 21, p. 12, 1902), and published in *Engineering News* (Vol. 50, p. 60, 1903), and in the *Engineering Record* (Vol. 48, p. 49, 1903).

(19) The insoluble residue shall be determined on a 1-gram sample which is digested on the steam bath in hydrochloric acid of approximately 1.035 specific gravity until the cement is dissolved. The residue is filtered, washed with hot water, and the filter paper and contents digested on the steam bath in a 5 per cent. solution of sodium carbonate. The residue is then filtered, washed with hot water, then with hot hydrochloric acid approximately of 1.035 specific gravity, and finally with hot water, then ignited and weighed. The quantity so obtained is the insoluble residue.

Determination of Specific Gravity

(20) The determination of specific gravity may be made with a standardized apparatus of Le Chatelier or other equally accurate form. Benzine (62° Baumé naphtha), or kerosene free from water, should be used in making the determination. The cement should be allowed to pass slowly into the liquid of the volumenometer, taking care that the powder does not adhere to the sides of the graduated

tube above the liquid, and that the funnel through which it is introduced does not touch the liquid. The temperature of the liquid in the flask should not vary more than 1° F. during the operation. To this end the flask should be immersed in water. The results of repeated tests should agree within 0.01.

(21) If the specific gravity of the cement as received is less than 3.10, a redetermination may be made as follows: Seventy grams of the cement are placed in a nickel or platinum crucible about 2 inches in diameter and heated for thirty minutes at a temperature between 419° C. and 630° C. After the cement has cooled to atmospheric temperature the specific gravity shall be determined in the same manner as described above. The cement should be heated in a muffle or other suitable furnace, the temperature of which is to be maintained above the melting-point of zinc (419° C.), but below the melting-point of antimony (630° C.). This maximum temperature can be recognized as a very dull red which is just discernible in the dark.

Determination of Fineness

(22) The Nos. 100 and 200 sieves shall conform to the standard sieve specifications of the Bureau of Standards, Department of Commerce and Labor.

(23) The determination of fineness should be made on a 50-gram sample which may be dried at a temperature of 100° C. (212° F.) prior to sifting. The coarsely screened sample should be weighed and placed on the No. 200 sieve, which, with the pan and cover attached, should be held in one hand in a slightly inclined position, and moved forward and backward in the plane of inclination, at the same time striking the side gently about 200 times per minute against the palm of the other hand on the upstroke. The operation is to be continued until not more than 0.05 gram will pass through in one minute. The residue should be weighed, then placed on the No. 100 sieve and the operation repeated. The sieves should be thoroughly clean and dry. Determination of fineness may be made by washing the cement through the sieve or by a mechanical sifting device which has been previously standardized with the results obtained by hand sifting on equivalent samples. In case of the failure of the cement to pass the fineness requirements by the washing method or the mechanical device, it shall be tested by hand.

Mixing Cement Pastes and Mortars

(24) The quantity of cement or cement and sand to be used in the paste or mortar should be expressed in grams and the quantity of water in cubic centimeters. The material should be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand be used, and a crater formed in the center, into which the proper percentage of clean water should be poured; the material on the outer edge should be turned into the crater by the aid of a trowel. As soon as the water has been absorbed, the operation should be completed by vigorously mixing with the hands for one minute and a half. During the operation of mixing, the hands should be protected by rubber gloves. The temperature of the room and the mixing water should be maintained as nearly as practicable at 21° C. (70° F.).

Determination of Normal Consistency

(25) The normal consistency for neat paste to be used in making briquettes and pats should be determined by the ball method, as follows:

(26) A quantity of cement paste should be mixed in the manner above described under Mixing Cement Pastes and Mortars, and quickly formed into a ball about 2 inches in diameter. The ball should then be dropped upon a hard, smooth, and flat surface from a height of 2 feet. The paste is of normal consistency when the ball does not crack and does not flatten more than one-half of its original diameter.

(27) Trial pastes should be made with varying percentages of water until the correct consistency is obtained.

(28) The percentage of water to be used in mixing mortars for sand briquettes is given by the formula:

$$y=\frac{2}{3}\frac{P}{n+1}+K,$$

in which y is the percentage of water required for the sand mortar; P is the percentage of water required for neat cement paste of normal consistency; n is the number of parts of sand to one of cement by weight; and K is a constant which for standard Ottawa sand has the value 6.5.

The percentage of water to be used for mortars containing three parts standard Ottawa sand by weight to one of cement is indicated in the following table:

Percentage of Water for Neat Cement Paste.	Percentage of Water for 1 to 3 Mortars of Standard Ottawa Sand.
18	9.5
19	9.7
20	9.8
21	10.0
22	10.2
23	10.3
24	10.5
25	10.7
26	10.8
27	11.0
28	11.2
29	11.3

Determination of Soundness

(29) Pats made of neat cement paste of normal consistency about 3 inches in diameter, ½ inch in thickness at the center, and tapering to a thin edge, should be kept in moist air for a period of twenty-four hours. One pat should then be kept in air and a second in water, at the ordinary temperature of the laboratory, not to vary greatly from 21° C. (70° F.), and both observed at intervals for at

least 28 days. A third pat should be exposed to steam at atmospheric pressure above boiling water for five hours.

DETERMINATION OF TIME OF SETTING

(30) The time of setting should be determined by the standardized Gilmore needles, as follows:

A pat of neat cement paste about 3 inches in diameter and ½ inch in thickness with flat top mixed at normal consistency should be kept in moist air, at a temperature maintained as nearly as practicable at 21° C. (70° F.). The cement is considered to have acquired its initial set when the pat will bear, without appreciable indentation, a needle $\frac{1}{12}$ inch in diameter loaded to weigh ¼ pound. The final set has been acquired when the pat will bear without appreciable indentation, a needle $\frac{1}{24}$ inch in diameter, loaded to weigh 1 pound. In making the test the needle should be held in a vertical position and applied lightly to the surface of the pat. The pats made for soundness test may be used to determine the time of setting.

TENSILE TESTS

(31) Tensile tests should be made on an approved machine. The test pieces shall be briquettes of the form recommended by the committee on uniform tests of cement of the American Society of Civil Engineers, and illustrated in Circular 33 of the Bureau of Standards. The briquettes shall be made of paste or mortar of normal consistency. Immediately after mixing, the paste or mortar should be placed in the molds, pressed in firmly by the fingers, and smoothed off with a trowel without mechanical ramming. The material should be heaped above the mold, and in smoothing off, the trowel should be drawn over the mold in such a manner as to exert a moderate pressure on the material. The molds should be turned over and the operation of heaping and smoothing repeated. Not less than three briquettes should be made and tested for each sample for each period of test. The neat tests are not considered so important as the sand tests. The briquettes should be broken as soon as they are removed from the water. The load should be applied at the rate of 600 pounds per minute.

STORAGE OF TEST PIECES

(32) During the first twenty-four hours after molding the test pieces should be kept in air sufficiently moist to prevent them from drying. After twenty-four hours in moist air the test pieces should be immersed in water. The air and water should be maintained as nearly as practicable at 21° C. (70° F.).

STANDARD SAND

(33) The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve.

(34) Sand having passed the No. 20 sieve shall be considered standard when not more than 2 grams pass the No. 30 sieve after one minute continuous sifting of a 200-gram sample.

(35) The No. 20 and No. 30 sieves shall conform to the standard sieve specifications of the Bureau of Standards, Department of Commerce and Labor.

Copies of the above specifications can be obtained upon application to the various navy pay officers or to the Bureau of Supplies and Accounts, Navy Department, Washington, D. C.

References: Y. and D., 5766, Nov, 20, 1903; Y. and D., 8789, June 6, 1905; Y. and D., 5766WM, Nov. 8, 1906; Y. and D., 5766, Nov. 22, 1907; Y. and D., 10015, Oct. 23, 1909; Navy Dept., 5471-77-1, Mar. 23, 1912; Y. and D., 10061-63-Ca.-AM, Mar. 25, 1912.

APPENDIX IX

SPECIFICATIONS (59C2) FOR CONCRETE AND CONCRETE MATERIALS AND MORTAR AND MORTAR MATERIALS *

The following specifications govern in the case of all contracts of which they form a part, except in so far as they may be modified by the special specifications for the work.

I. MATERIALS FOR CONCRETE AND MORTAR

(*a*) *Sand for concrete* shall be clean and siliceous, and shall be a well-graded mixture of coarse and fine grains, with the coarse grains predominating. It shall be free from clay, loam, mud, organic matter, or other impurities. It shall be screened to remove all particles not passing a ¼-inch mesh screen, unless, in the opinion of the officer in charge, the proportion of particles above ¼ inch is so small that the sand will perform its functions in the concrete without screening. Sand for concrete may contain not more than 5 per cent of silt when measured by volume by shaking a sample of the material with water in a test-tube and allowing it to subside. Crusher dust or screenings passing a ¼-inch mesh screen may be combined with and measured as sand, but not more than one-third of the sand in any one batch may be of this material, unless it can be shown that the sizes of the particles are practically in the same proportion as in the most suitable grades of natural sand. Sample of the sand may be submitted to the officer in charge for approval before bidding if desired.

(*b*) *Sand for mortar* shall be clean and siliceous, and shall be composed of grains of varying size. It shall be free from clay, loam, mud, salt, organic matter, or other impurities, and shall also be free from silt. It shall be screened, if necessary, to remove all particles not passing through a ¼-inch mesh screen. If joints in the brickwork are too thin to allow the use of particles of ¼-inch size, then the screen used shall be of such a mesh as to exclude particles not suitable for use in the particular thickness of joint in use.

(*c*) *Broken stone.*—Crushed granite, trap, gneiss, or other equally suitable rock, may be used for concrete. It shall be free from clay, loam, mud, organic matter, and other impurities. Fine crushed stone passing a ¼-inch mesh screen may be combined with and measured as sand for concrete, as specified under "Sand for Concrete."

(*d*) *Gravel.*—Screened gravel may be used in lieu of broken stone where the latter is specified. Gravel shall be composed of hard, durable stone, and shall be clean, free from slaty or soft stones, clay, loam, mud, organic matter, and other impurities.

* Issued by the Navy Department, March 25, 1912.

(*e*) *Sizes of broken stone and gravel.*—Materials shall be screened to size and shall be run of the crusher or of the bank between the limits given. The particles shall vary in size between the upper and lower limits in order that the voids may be a minimum. For foundations or mass concrete the stone shall pass a 2-inch screen and be retained on a ¼-inch screen; for reinforced concrete the stone shall pass a 1-inch screeen and be retained on a ¼-inch screen, but when the distance between the reinforcing strands is less than 2 inches the upper limit shall not be over ¾ inch.

(*f*) *Water.*—Only fresh and clean water shall be used in mixing concrete and mortar.

(*g*) *Cement* shall be in accordance with the United States Government specifications for Portland cement as issued by the Navy Department. Cement furnished as a part of a public works contract shall be stored by the contractor, immediately upon delivery, in a suitable weather-tight and properly ventilated place, having a floor raised above the ground. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

(*h*) *Lime* shall be of the best quality, fat, well burned, and perfectly fresh lump lime, of a brand well known to the trade or an established brand of hydrated lime. All lime shall be shipped to the site in barrels bearing the name and label of the brand.

II. MEASUREMENT OF MATERIALS

(*a*) *Cement* shall be measured by weight and not by volume, and for the purpose of proportioning concrete or mortar 100 pounds shall be taken as the equivalent of 1 cubic foot. A sufficient number of the bags or barrels shall be actually weighed to insure the required amount in each batch.

(*b*) *Sand, broken stone, or gravel* shall be measured by volume for each batch in boxes, barrels, or other equally effective measuring devices approved by the officer in charge.

III. PROPORTIONING MATERIALS IN CONCRETE

(Method A, fixed volumes of sand and stone.)

(*a*) *Mass concrete* (1-3-6).—Foundations for buildings, including wall foundations, column piers, curtain walls, retaining walls in earth, abutments, wall footings, concrete foundations, and mass concrete similar to these shall be composed of 1 part cement (allowing 100 pounds to the cubic foot), 3 parts by volume of sand, and 6 parts by volume of broken stone or gravel.

(*b*) *Reinforced concrete* (1-2-4) in columns, beams, slabs, walls, etc., shall be composed of 1 part cement (allowing 100 pounds to the cubic foot), 2 parts by volume of sand, and 4 parts by volume of broken stone or gravel.

(Method B, fixed volume of stone and variable volume of sand.)

(*c*) *Proportions of concrete* under Method B, which is to be used only when specially required by the specifications for the work, shall be as follows:

Class A.—One part cement (allowing 100 pounds to the cubic foot) to 6½ parts by volume of broken stone combined with a variable proportion of sand.

Class B.—One part cement (allowing 100 pounds to the cubic foot) to 4½ parts by volume of broken stone combined with a variable proportion of sand.

(*d*) *Determining amount of sand.*—It is the intention with the given amounts of cement and broken stone to secure concrete as dense as possible. The amount of sand to be added will therefore depend on the actual character of the sand and broken stone, and the number of cubic feet of sand to be added will be determined by frequent experiments and tests of the material as actually delivered and accepted during the progress of the work. The proportion will be established from time to time for each class of aggregate used and will not be changed during any period of twenty-four hours, unless the contractor desires to use materials of different characteristics, and the proportion determined at any one time shall continue to be used until the next determination is made and the contractor has been ordered to change the previous proportions.

(*e*) *Tests for amount of sand* shall be conducted as follows: To a fixed volume (not less than 5 cubic feet) of dry stone, which shall be a representative sample, add a fixed volume of dry sand. Mix these very thoroughly, so as to fill all the voids as uniformly as possible, and then measure and weigh the resulting mixture. Repeat the experiment with varying amounts of sand. That proportion of sand which gives the heaviest mixture per unit of volume shall be used in preparing the concrete. The labor and materials for the experiments shall be furnished by the contractor, but the experiment shall be conducted under immediate supervision of the officer in charge.

(*f*) *Run of crusher stone.*—The specifications elsewhere require the screening out of particles of broken stone and gravel below ¼ inch in size. Under method "B" of proportioning materials, this will not be necessary if it can be shown by tests that the amount of fine material remaining in the stone or gravel is such that with the addition of sand the resulting total of fine material below ¼ inch size will be at least equal to the most suitable grades of natural sand. Tests must be made to determine that there is not an excess of fine material beyond that required to give the densest possible aggregate. Permission to use run of crusher stone as described in this paragraph must be previously obtained from the officer in charge of the work.

IV. MIXING MATERIALS

(*a*) *Method of mixing.*—Concrete shall be mixed by hand or by a mechanical mixer of a type to be approved by the officer in charge.

(*b*) *Presence of inspector.*—Concrete may be mixed and placed only in the presence of an inspector, and the contractor or his agent shall give due and ample notice to the officer in charge when mixing is to be commenced. The officer in charge may reject any concrete mixed or placed without the presence of an inspector when such notice has not been given.

(*c*) *Hand mixing.*—For mixing by hand use only strong, water-tight, well-built platforms, large enough to provide space for the partial simultaneous mixing of two batches, which shall not consist of more than 1 cubic yard each, shall be used. The sand and cement in specified proportions shall be spread in layers on the mixing platform, the sand at the bottom, and shall then be thoroughly and evenly mixed dry until of uniform color without streaks. Water shall then be added and thoroughly and uniformly incorporated. The sand and cement mixture shall then be placed on the broken stone, which shall have been pre-

viously wet down and placed in a layer on the board adjoining the cement and sand. The mixture shall then be worked up and turned over at least three times, not including shoveling from the platform to place of deposit or into the vehicle transportation. The material on the shovel shall be completely turned over and not merely dropped from the shovel. The number of turnings shall be more than three if necessary to produce a thoroughly mixed concrete of uniform consistency throughout. The use of a rake is permitted in mixing the sand and cement, but is forbidden after the stone has been added. The amount of water shall be such as to give the consistency specified elsewhere, and special care must be taken not to exceed the proper amount. Should an excess of water be added inadvertently, the batch shall be remixed with sufficient additional cement to take up the water. The details of the method of hand mixing may be varied from the above by permission of the officer in charge, but the results must be substantially the same as produced by the method described.

(*d*) *Machine mixing*, if efficiently done, is ordinarily to be preferred to hand mixing. The mixer shall be tested at the beginning of the work to determine the most efficient size of batch and method of use. The materials for each batch shall be carefully and accurately measured and introduced into the mixer. Water shall be introduced into the mixer during the process of mixing, and the amount shall be accurately measured to give the required consistency as determined by experience with previous batches. Should too much water be added inadvertently, the batch shall be remixed with sufficient additional cement to take up the water. The machine shall be of the batch type, and shall produce a concrete thoroughly mixed and of uniform mixture and consistency superior to or at least equal to that produced by the hand mixing above described. The type of machine selected is subject to approval by the officer in charge. A machine of the continuous or of the gravity type shall not be used unless specially permitted by the specifications for the work.

(*e*) *Consistency.*—A medium or quaking mixture of tenacious jelly-like consistency which quakes under light ramming shall be used for all concrete except reinforced concrete. For reinforced concrete a very wet or mushy mixture shall be used, such that it will flow into the forms and around the reinforcement, but not so wet as to allow the materials to separate in handling or transporting. For reinforced roof concrete, the mixture shall be slightly dryer.

V. HANDLING AND PLACING CONCRETE

(*a*) *Handling.*—Concrete shall be conveyed and deposited in such a manner that there may be no separation of the different ingredients. Any concrete which has commenced to set before placing shall be rejected and immediately removed from the work and wasted in such manner as may be directed by the officer in charge.

(*b*) *Placing.*—The specified consistency of the concrete will require only light tamping or spading. Along the face of the forms the concrete shall be spaded in such a manner as to force back the larger particles and bring the mortar to the surface of the form in order to avoid pits and irregular concrete after removal of the forms. Except in continuous laying, the concrete shall not be deposited upon that which has been laid less than twelve hours. The surface of each layer which is not thoroughly clean or which has been in place more than twelve

hours shall be thoroughly cleaned and wet, and then broomed with one-to-one cement before depositing the succeeding layer.

(*c*) *Joints in reinforced concrete.*—All slabs shall be jointed over the centers of beams and girders. Beams and girders shall be built in sections continuously from the center line of one girder or column to the next, the deeper girder being recessed to receive the shallower one unless both are built simultaneously. Where both beams and girders intersect over the same column, the joints shall run diagonally across the column so as to secure proper bearing for each.

(*d*) *Protection of concrete after depositing.*—Concrete shall be protected from injurious action by the sun, heavy rains, currents of water, frost, or mechanical injury. In dry weather it shall be wet down frequently.

(*e*) *Freezing weather.*—Concrete shall not be deposited in freezing weather nor at such times as it is likely in the opinion of the officer in charge to be subjected to freezing weather within twenty-four hours after being deposited, except on the written authority of the officer in charge, and then only under such restrictions and in such manner, as he may prescribe.

(*f*) *Depositing under water.*—Concrete shall not be deposited under water unless distinctly permitted by the specifications for the work. Concrete shall at all times be protected from running water.

(*g*) *Forms.*—All forms shall be waterproof to prevent leakage of cement and of a substantial character, well braced, and sufficiently strong to hold the face of the concrete true to line. The design and character of forms shall be subject to approval by the officer in charge, and if not approved by him shall be remodeled and improved. If the forms are held together by bolts or wires, they shall be so made that no iron will be left exposed on the faces of the finished work. All forms for exposed work shall be surfaced one side and free from defects affecting the finished appearance of the concrete. Forms shall remain in place long enough to allow the concrete to properly set, as determined by the officer in charge.

VI. MORTARS

(*a*) *Cement mortar* shall be composed of 1 part Portland cement and 2 parts sand. The sand and cement in specified proportions shall be spread in layers on a closed-in platform or tight mortar box, the sand at the bottom, and then thoroughly and evenly mixed dry until of uniform color without streaks. Water shall then be added and thoroughly and uniformly incorporated with the mixture until it is of the proper consistency for use with the work for which it is intended.

(*b*) *Lime paste* shall be prepared in a tight box by adding to the lime sufficient water to thoroughly slack the lime without "burning" or "chilling." The slacked lime while still warm shall be sifted or run through a sieve having meshes ⅛ inch square in order to remove all particles of unslacked or partially slacked lime. After cooling it shall be stored and properly protected. Lime paste for mortar shall stand one week before using.

(*c*) *Lime mortar* shall be composed of 2 parts of cement (allowing 100 pounds to the cubic foot), 1 part of lime paste by volume, and 6 parts of sand by volume. The lime paste and sand shall be thoroughly mixed not less than twenty-four hours before using, and 6 parts of the mixture shall be taken to mix with the 2 parts of cement. The cement shall be added only in small quantities sufficient for immediate use. Retempering of mortar in which cement has already begun

to set will not be permitted. The mortar must be used so that it will be in place within the limit of time of the initial setting of the cement.

Copies of the above specifications can be obtained upon application to the various navy pay offices or to the Bureau of Supplies and Accounts, Navy Department, Washington, D. C.

References: Y. and D., 9113-S-T, Mar. 1, 1912; Dept., 5471-88, Mar. 2, 1912; S. and A., 118159.

APPENDIX X

SPECIFICATIONS FOR CREOSOTING PILING (WATERSOAKED) *

THE piling to be treated must be Douglas fir, thoroughly sound and free from all defects calculated to impair its value; they must be perfectly straight from end to end, free from bark, Teredo, Limnoria or other seaworm holes, also from barnacles and similar attachments. They must be cut from mature stock and show an even taper from butt to point. The butts shall not be less than a full fourteen (14″) inches in diameter, nor less than nine (9″) inches in diameter at the point.

Each cylinder charge must be made up of piles which have been in the water as nearly as possible the same length of time; nor must they have been so long therein as to cause deterioration or damage of any kind.

After the piles are placed in the cylinder they must be immersed in creosote oil of the quality specified below, of a temperature ranging between 160 and 170° F., and kept covered during the entire boiling period under at least 4 inches of oil at the shallowest place. The engineer on duty must from time to time during the boiling satisfy himself by bleeding the cylinder that such is the case.

After filling the cylinder with oil, steam must then be regulated through the heating coils, so that the temperature within the cylinder is kept gradually rising as fast as the condensation will permit until 220° F. is reached; after which the steam pressure must only be such as to maintain a regular and constant temperature within the cylinder with 220° as the minimum and 225° F. the maximum until such time as the amount of condensation per cubic foot per hour collecting in the hot well of the condenser shows the interior of the wood to be thoroughly dry, when the steam pressure in the coils should be released, and the cylinder filled up with creosote oil from the storage or working tanks, of a temperature ranging between 160 and 170° F., then pump pressure applied until the gage shows 5 pounds pressure in the cylinder (this to insure the fact of the cylinder being actually full), after which the connection between the storage tank and cylinder should be closed and the connection between measuring tank and cylinder opened. Additional pressure must then be applied until the piling has taken the proper amount of oil, forced in under such conditions as will insure its complete retention in the wood after drip is over, and figured at the weight of the dry oil per gallon at 100° F., the cylinder doors may then be opened provided the temperature within is reduced to below 200° F.

* F. D. Beal, Manager, St. Helens Creosoting Co., Portland, Ore.

After treatment, the piling must be free from all heat checks, water bursts and other defects due to inferior treatment which would impair usefulness or durability for the purposes intended. Piles when bored midway between the ends must show no moisture in the center, and the borings beyond the oil penetration must retain their natural elasticity and strength.

GREEN OR FRESHLY CUT AND SEASONED PILING

Green or freshly cut piles delivered at the treating plant on cars, or any which have not been lying in the water, must be treated in the manner prescribed.

Thoroughly seasoned piles must be treated in the manner prescribed.

No piling in these two classes must be mixed together and treated in the same charge, and none in either of these two classes should be treated which is not at the time free from rot, and in proper condition for use after treatment as far as splits or breaks are concerned; if any such is received from the mills it should not be treated unless the inspector directs it to be done.

After the piles are placed in the cylinder, it must be immersed in creosote oil of a temperature ranging between 160 and 170° F., and kept covered during the entire boiling or heating period under at least 4 inches of oil at the shallowest place; the engineer on duty must from time to time during the boiling satisfy himself by bleeding the cylinder that such is the case.

In the case of green or freshly cut piling, steam must thereafter be regulated through the heating coils so that the temperature within the cylinder is kept gradually rising as fast as the condensation will permit until 212° F., is reached, with 215° F. as the maximum; after which the steam pressure must only be such as to maintain a regular and constant temperature within the cylinder between these figures, until such time as the amount of condensation per cubic foot per hour collecting in the hot well of the condenser shows the interior of the wood to be thoroughly dry, when steam pressure in the coils should be released.

In the case of thoroughly seasoned piling, the temperature of the oil in the cylinder must be allowed to rise slowly and steadily until 190° F. is reached, with 192° F. as the maximum; and kept between these figures until such time (depending upon the dimensions) as the interior of the wood shall have become sufficiently warmed up to enable it to take the required amount of oil, when the steam pressure in the coils should be released.

The cylinder should then (in each case) be filled up with creosote oil from the storage or working tank, of a temperature ranging between 160 and 170° F., and pressure from the pump applied until the gage shows 5 pounds pressure in the cylinder (this to insure the fact of the cylinder being actually full), after which the connection between the storage tank and cylinder should be closed, and the connection between measuring tank and cylinder opened. Additional pressure must then be applied slowly and steadily until the piling has taken the proper amount of oil forced in under such conditions as will insure its complete retention in the wood after drip is over, and figured at the weight of the dry oil per gallon at 100° F., the cylinder doors may then be opened provided the temperature within is below 200° F.

After treatment the piling must be free from all heat checks, water bursts, and other defects due to inferior treatment, which would impair usefulness or durability for the purposes intended.

FIR SAWED MATERIAL

Seasoned, and green or freshly sawed material must not be mixed together and treated in the same charge, and none should be treated, which is not at the time free from rot, and in the proper condition for use after treatment so far as splits or breaks are concerned, if any such is received from the mills, it should not be treated unless the inspector directs it to be done.

Square timber must not be treated in the same charge with planking, nor ties with planking, and sufficient 1-inch strips must be placed between each tier, with from ½ to 1 inch space left between each piece, so that the oil can have free access to all surfaces.

After the material is placed in the cylinder, it must be immersed in creosote oil of a temperature ranging between 160 and 170° F., and kept covered during the entire boiling or heating period under at least 4 inches of oil at the shallowest place; the engineer on duty must from time to time during the boiling satisfy himself, by bleeding the cylinder, that such is the case.

In the case of green or freshly sawed material, steam must thereafter be regulated through the heating coils so that the temperature within the cylinder is kept gradually rising as fast as condensation will permit until 212° F. is reached, with 215° F. as the maximum, after which the steam pressure must only be such as to maintain a regular and constant temperature within the cylinder between these figures, until such time as the amount of condensation per cubic foot per hour collecting in the hot well of the condenser shows the interior of the wood to be thoroughly dry, when steam pressure in the coils should be released.

In the case of thoroughly seasoned timber, the temperature of the oil in the cylinder must be allowed to rise slowly and steadily until 190° F. is reached, with 192° F. as the maximum, and kept between these figures until such time (dependent upon the dimensions) as the interior of the wood shall have become sufficiently warmed up to enable it to take the required amount of oil, when the steam pressure in the coil should be released.

The cylinder should then (in each case) be filled up with creosote oil from the storage or working tank, of a temperature ranging between 160 and 170° F. and the pressure from the pump applied until the gage shows 5 pounds pressure in the cylinder (this to insure the fact that the cylinder is actually full), after which the connection between the storage tank and the cylinder should be closed, and the connection between the measuring tank and the cylinder opened. Additional pressure must then be applied slowly and steadily until the material has taken the proper amount of oil, forced in under such conditions as will insure the complete retention in the wood after drip is over, and figured at the weight of the dry oil per gallon at 100° F.; the cylinder doors may then be opened provided the temperature within is below 200° F.

After treatment the material must be free from all heat checks, water bursts, and other defects due to inferior treatment, which would impair usefulness or durability for the purposes intended.

CREOSOTE OIL

The oil used in treating shall be the best obtainable grade of coal-tar creosote; that is it must be a pure product of coal-tar distillation, and must be free from

admixture of oils, other tars, or substances foreign to pure coal-tar; it must be completely liquid at 38° C., and must be free from suspended matter; the specific gravity of the oil at 38° C., must be at least 1.03. Creosote oil, when distillation is carried on as laid down in the American Railway Engineering and Maintenance of Way Association, Vol. 9, page 708, shall give no distillate below 200° C. not more than 5 per cent. below 210° C., not more than 25 per cent. below 235° C. and the residue above 355° C. if it exceed 5 per cent. in quantity, must be soft. The oil shall not contain more than 3 per cent. of water.

GENERAL CLAUSES

All material shall be treated to the entire satisfaction of the purchaser's inspector, he being allowed full access at all times to the facilities used in the treatment while it is in progress. When recording thermometers are installed on treating plants the inspector shall have access to same in order to check temperatures during treatment. He shall also be furnished with facilities for taking daily inventories on creosote oil in order to ascertain the exact amount injected in the timber; but the fact of any inspector being at the plant shall not relieve the treating company's officials from the responsibility of seeing that the treatment of all material is properly and carefully done, with the agreed penetration of oil in each case, based on the contract amount.

Before each cylinder charge is disposed of, the depth and character of the penetration must be ascertained by boring one or more auger holes, after wood has cooled, in as many pieces of each class of material as may be necessary for the purpose, and such pieces as are not found to be fully treated in accordance herewith must be returned to the cylinder with a subsequent charge for further treatment without extra cost therefor; should more than 10 per cent. of the total number of pieces treated be found defective, the entire charge must be so returned. No material must be removed from the treating yard until all auger holes are tightly plugged with creosoted plugs.

The intent of these specifications is that the wood when it comes out of the cylinder and after all drip is over, shall contain the full weight of oil to the cubic foot, forced in at such pressure, and under such conditions as to enable the wood cells and fiber to retain it permanently.

The pressure gages and thermometers must be compared and tested at frequent intervals with standard test appliances kept on hand for that purpose, and all differences corrected.

Competent and experienced engineers shall be in charge of the treatment night and day, and required to make frequent examinations of the temperature during the boiling, especially when the maximum heat is being applied; the thermometers being located so that they will correctly reflect the heat conditions within the cylinders, and at the same time be convenient to get at.

In handling the material after treatment, sharp dog or cant hooks must not be used in any way whereby the full protection of the treatment is likely to be lessened; in the case of piling they must be used in the space within 2 feet from the large end and 6 feet from the point. Any material broken or otherwise damaged in treatment or by careless handling, while in the treating company's care and until delivered to the purchaser's destination as per contract, will be rejected, and the treating company must substitute new material therefor or the cost will be charged to the treating company.

Facilities for testing the grade of the oil to ascertain that it is in accordance with the above specifications shall be furnished by the treating company and all necessary facilities for testing the same shall be provided, and kept at the plant by the treating company.

The treatment under these specifications will require 12 pounds of creosote oil per cubic foot.

APPENDIX XI

PILING DATA FOR SPECIAL CASES

PILES AS COLUMNS. The use of piles to reach a hard stratum to carry the load, where the piles are driven through a soft material, causes the piles to act as timber columns, and no vertical support should be considered from a slight possible friction on the surface of the pile. Where the material is packed and moderately firm, it may be considered as affording a lateral support for the pile, and the load computed by multiplying the area in square inches of the pile by the safe crushing value for the class of timber used, in pounds per square inch. This may be taken as follows for a factor of six:

Oak	750 pounds	Constant (a)
Southern pine	850 pounds	Constant (a)
Douglas fir	1000 pounds	Constant (a)
Spruce	700 pounds	Constant (a)
Hemlock	700 pounds	Constant (a)
Cedar	700 pounds	Constant (a)
Redwood	700 pounds	Constant (a)
White pine	600 pounds	Constant (a)

This may also be considered to be safe for columns where the length divided by the diameter is under 15, but where the pile is in water or the material is soft mud, very wet sand, or other material which does not afford support sidewise, and where the ratio of length to diameter of such piles is over 15, then the strength in pounds per square inch must be computed from the formula:

For factor of four:

$$P = 1500 - 18\frac{l}{d} \text{ (Douglas fir).}$$

For factor of six:

$$P = 1000 - 12\frac{l}{d} \text{ (Douglas fir).}$$

For factor of eight:

$$P = 750 - 9\frac{l}{d} \text{ (Douglas fir);}$$

P = safe load in pounds per square inch;
l = length unsupported in inches;
d = average diameter of part unsupported.

For variation of timber other than Douglas fir, the value derived from the formula must be varied in proportion to the preceding value or Constant (a) for the kind of timber being employed.

Piles in Bending. The use of piles in wharves or similar structures is often of such a character as to cause transverse loading, and in soft mud or sand which flows or slides bodily, a similar or more severe bending may result. The fiber stress must be calculated by considering the pile as loaded along its length by a fluid pressure from liquid mud assumed as weighing 100 pounds per cubic foot, acting on the projected area of the pile. This unit stress plus the direct load in pounds per square inch, must not exceed the unit value derived from the preceding formula for a factor of safety of four.

When the value of the combined units does exceed this allowable value, then more piles must be used or spur piles and cribbing employed in such a manner as to relieve the pressure on the piles. Where the foundation bed is liable to move bodily, then the use of spur piles becomes an absolute necessity, or else other protective and retaining works of timber or concrete must be constructed.

Pile Bracing. The most satisfactory method of bracing pile foundations is with spur piles, or piles driven at an angle of from 30 to 45° with the vertical. Where all of the piles are to be capped with a bed of concrete, this will sufficiently tie in the brace piles; but where timber caps are to be used on top of the piles, the brace piles must be framed into the adjacent vertical piles by framing the top of the brace pile to fit a notch near the top of the vertical pile, making the notch far enough below the top to give ample shearing section, and bolting them together with enough bolts to hold them rigidly in position.

The method of simply cutting off the spur piles to fit under the caps and fastening them with drift bolts is to be avoided if possible, but where similar detail is necessary the cap should be made of two pieces gained into sides of the vertical pile, bolted with at least two bolts and the brace pile framed to fit under the cap.

Where the piles project for some distance above the ground or above the bottom, as in the case of wharves, plank or timber cross-bracing is often used, and the horizontal timbers should be gained into piles, with two bolts at each fastening. The piles should be flattened where the cross-bracing connects, and not less than two bolts used for fastening. The cuts in the piles must all be painted with heavy white lead for work in the dry, or with hot creosote oil where they are covered by salt tidewater; and the bracing itself treated with hot carbolineum for work in the dry; and creosoted when they will be covered with salt water where there are teredo.

Pile Spacing. The spacing of piles center to center is an important matter, as when driven too close to each other they will either crowd one another out of line or position, or in soft and spongy material, the pile being driven may force the adjacent ones up out of the ground. They can generally be spaced much closer without bad results when steam pile hammer is used, or when water jets are employed with either type of hammer.

The minimum spacing of clusters of piles or of adjacent rows of piling may be taken at 2 feet 6 inches centers of piling for piles with 12 inches or less butt diameter; at 2 feet 9 inches centers of piling for piles with about 14 inches butt diameter; at 3 feet 0 inch centers of piling for piles with about 16 inches butt diameter; and at 3 feet 6 inches centers of piling for piles with about 20 inches butt diameter.

There is sometimes a necessity for spacing the piles opposite in the adjacent rows, such as where they are to be capped with timber caps or where they are to

support parallel rows of steel beams or girders, but usually they are to be capped with concrete, so that the piles in adjacent rows can be staggered, which makes it possible to drive the rows closer together and still have the diagonal distance enter to center of piles as great as necessary. This also makes it easier to drive and properly fasten spur piles, as with the piles opposite it would sometimes be necessary to make wider spacing to admit them.

Pile Layouts. The piling plan of a pier or building is one which should receive the most careful study, as upon it will depend in many cases the integrity of the foundations, and consequently of the structure itself. The layout also to a great extent determines the economy of the foundations, as the closest spacing of piles that is possible will usually be of enough spread, or cover enough area for safety; and this will make the quantity of excavation and concrete a minimum. The minimum spacing is not, however, always desirable and seldom necessary for the piling plans of special structures, such as towers or very high buildings that are in isolated locations.

Pile Loads for Varying Lengths of Piling

The proportionate safe loads for piles in packed sand, based on a safe load per square foot on the frictional area, would be as follows:

40 feet long, 12-inch butt, 9-inch top...........	16 tons
45 feet long, 14-inch butt, 9-inch top...........	20 tons
50 feet long, 16-inch butt, 9-inch top...........	25 tons
55 feet long, 18-inch butt, 8-inch top...........	28 tons
60 feet long, 20-inch butt, 8-inch top...........	33 tons
65 feet long, 22-inch butt, 8-inch top...........	36 tons
70 feet long, 24-inch butt, 8-inch top...........	40 tons

This would indicate an increased factor of safety for the longer and larger piles, if the maximum of 25 tons as previously given is adhered to, which owing to the great uncertainty of the frictional support and the sidewise support, would in any event usually seem advisable. However, should a larger load than 25 tons per pile be used, the designer must make sure that such a load will not crush the head of the pile, or a timber resting upon the pile head.

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